Appendix 5-B-1 "A" Dimension for Prestressed Girder Bridges

(For Simple Span or Continuous P.S. Bridges)

Definitions:

S = Span length (ft.)
L = Vertical curve length (ft.)
G = Algebraic diff. in profile tangent grades (%)
R = Horizontal curve radius to girder per next sheet (ft.)
W = Girder top flange width (inches)
m = Deck crown or super slope (ft./ft.)

Note: The following assumes that sag breaks in curb line profiles due to super transitions will occur @ Piers so as not to require any increase "A."

"A" Dimension (At Piers only)

(Slab Thickness + $^3/_4''$) fillet	= +	(Normally $8^1/_4''$)
① Excess Girder Camber Allowance	= + ⊛	
Top Flange Width Effect = $W \times \frac{m}{2}$	= +	
Horiz. Curve Effect = $\frac{1.5\ S^2 m}{R}$	= +	
Vert. Curve Effect = $\frac{1.5\ GS^2}{100L}$	= {+ for Sag Vert. or - for Crown Vert.)	
Round "A" to nearest $^1/_4''$	Total "A"	

(See minimums below) May make shorter span critical.

{ Use "A" = (Slab thickness + $^3/_4''$) + Top Flange Width Effect) Min.
Use "A" = 9″ Min. where Drain Type 5 crosses girder.

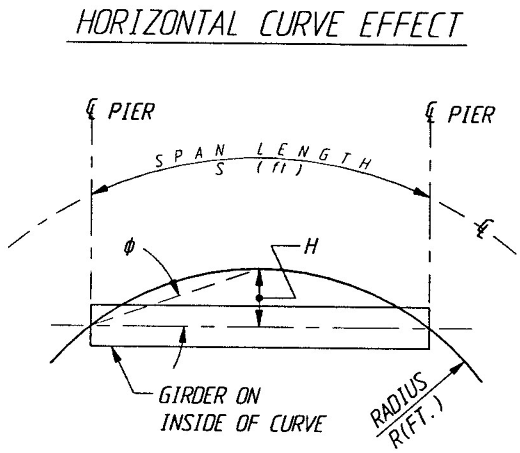
The basic attempt is to have the top of girder not higher than $^3/_4''$ below the bottom of slab at the center of the span. This provides that the actual girder camber could exceed the calculated value by $1^3/_4''$ before the top of the girder would start interfering with the slab steel.

① Allowance for the amount the girder camber, at time of slab casting,"D" dimension from Girder Schedule Table.

⊛ Use 2.50 @ preliminary plan stage to determine vertical clearance. Note in left margin of Layout: "A" Dimen. = "X" (not for design).

Use value from deflection program results to determine "A" Dimen. to use for design.

Horizontal Curve Effect:



$$\phi = \frac{5{,}730}{R} \times \frac{S}{400} \times m$$

$$\tan \phi = \frac{5{,}730Sm}{400R} \times \underset{\text{(approx.)}}{0.01746} \qquad \tan 1°$$

$$H = \frac{573Sm}{4R} \times 0.01746 \times \frac{S}{2} \times 12$$

$$H \quad \frac{1.5 \times S^2}{R} \times m \text{ (inches)} \longleftarrow$$

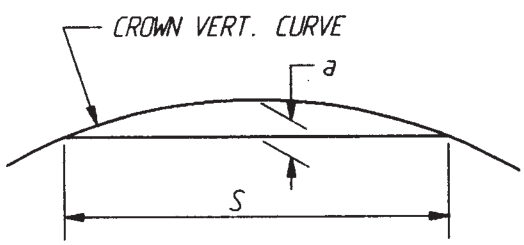
(approx.)

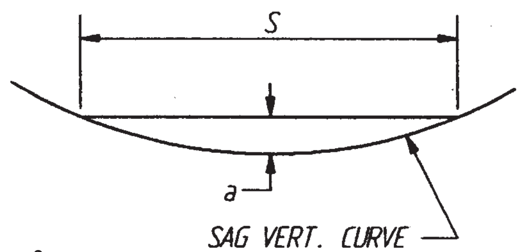
Vertical Curve Effect:

Algebraic difference in profile tangent grades = G (%)

Vertical curve length = L (ft)

Span Length = S (ft)





$$K = \frac{100G}{2L} \qquad a = K \times \frac{S^2}{40{,}000} \times 12 = \underline{G} \times \frac{S^2}{400} \times 12$$

$$a = 1.5 \times \frac{G}{100\,L} \times S^2 \text{ (inches)} \longleftarrow$$

Check for excess pad at ℄ span

For bridges which are on sharp crowned vertical curves, the pad at ℄ span can become excessive to the point where the girder and diaphragm stirrups (based on the "A" dimension) are too short to bend into the proper position. This is a problem on bridges with spans in excess of 100 feet and a total grade change of 10 percent on a 900-foot vertical curve. The effect of girder cambers which are less than the calculated values tends to add to this error. The "A" dimension @ ℄ centerline span shall not be less than:

"A" ℄ = "A" pier ± Vertical Curve effect + Excess Camber — (D) Screed Camber (C)

A value for "D" of 1 inch less than that shown on the Girder Schedule Table should be used to accommodate the worst case of camber variation.

A correction should be made to the stirrup lengths if the value of A ℄ exceeds A by more than 2 inches.

Preapproved Post-Tensioning Anchorages

The following are the anchorages approved by the Washington State Department of Transportation. The majority of these anchorages have been approved and accepted by WSDOT on the bases of tests done by suppliers for various state and local jurisdictions outside the state of Washington.

VSL Corporation (Owned by DYWIDAG Systems International)		
Anchorage	Type	Maximum Number of Strands
E5-12	Bearing Plate	12 ½-inch strands
E5-19	Bearing Plate	19 ½-inch strands
E5-22	Bearing Plate	22 ½-inch strands
E5-31	Bearing Plate	31 ½-inch strands
E5-37	Bearing Plate	37 ½-inch strands
E6-12	Bearing Plate	12 0.6-inch strands
E6-19	Bearing Plate	19 0.6-inch strands
E6-22	Bearing Plate	22 0.6-inch strands
E6-31	Bearing Plate	22.06-inch strands
EC5-31	Casting With External and Intermediate	31 ½-inch strands
EC5-27	Casting With External and Intermediate	27 ½-inch strands
EC5-19	Casting With External and Intermediate	19 ½-inch strands
EC5-12	Casting With External and Intermediate	12 ½-inch strands
SO6-4	Casting With External Flange (for Bearing)	4 0.6-inch strands used for dock (or slab) post-tensioning
ACS-28.5	Bearing Plate	28 ½-inch strands
ACS-24.5	Bearing Plate	24 ½-inch strands
ACS-22.5	Bearing Plate	22 ½-inch strands
C-22.5	Casting With External and Intermediate	22 ½-inch strands
ES5-12	Single Plane Anchorage	12 ½-inch strands
ES5-19	Single Plane Anchorage	19 ½-inch strands
ES5-31	Single Plane Anchorage	31 ½-inch strands
ES6-7	Single Plane Anchorage	7 0.6 inch strands
ES6-12	Single Plane Anchorage	16 0.6 inch strands
ES6-19	Single Plane Anchorage	19 0.6 inch strands
ES6-22	Single Plane Anchorage	22 0.6 inch strands

AVAR Post-tensioning Systems		
Anchorage	Type	Maximum Number of Strands
SP 12.5	Single Plane System	12 ½-inch strands
SP 19.5	Single Plane System	19 ½-inch strands
SP 27.5	Single Plane System	27 ½-inch strands
SP 37.5	Single Plane System	37 ½-inch strands
MP 12.5	Multiple Plane System	12 ½-inch strands
MP 22.5	Multiple Plane System	22 ½-inch strands
MP 34.5	Multiple Plane System	34 ½-inch strands
C 12.5	Single Plane System	12 ½-inch strands
C 19.5	Single Plane System	19 ½-inch strands
C 27.5	Single Plane System	27 ½-inch strands

DYWIDAG Systems International		
Anchorage	Type	Maximum Number of Strands
MA12-0.5”	Mutiplane Anchorage	12 ½-inch strands
MA 9-.06”	Mutiplane Anchorage	9.6-inch strands
MA 15-.05”	Mutiplane Anchorage	15 ½-inch strands
MA 120-0.6”	Mutiplane Anchorage	12.6-inch strands
MA 20-.05”	Mutiplane Anchorage	20 ½-inch strands
MA 15-.6”	Mutiplane Anchorage	15.6-inch strands
MA 27-0.5”	Mutiplane Anchorage	27 ½-inch strands
MA 19-.06”	Mutiplane Anchorage	19.6-inch strands
MA 37-0.5”	Mutiplane Anchorage	37 ½-inch strands
MA 27-.06”	Mutiplane Anchorage	27.6-inch strands
MA 15-.5”	Mutiplane Anchorage	27 ½-inch strands
MA 15-.5”	Mutiplane Anchorage	27 ½-inch strands
Bar Anchorages 1-inch thread bars through 1 3/8 at fu of 150 ksi only.		

* Total strands include all bottom strands and top strands

** Strands are not developed fully, designer should check capacities and span lengths

The following listed bridge widenings are included as aid to the designer. These should not be construed as the only acceptable methods of widening; there is no substitute for the designer's creativity or ingenuity in solving the challenges posed by bridge widenings.

Bridge	SR	Contract No.	Type of Bridge	Unusual Features
NE 8th Street U'Xing	405	9267	Ps. Gir.	Pier replacements
Higgins Slough	536	9353	Flat Slab	
ER17 and AR17 O-Xing	5	9478	Box Girder	Middle and outside widening.
SR 538 O-Xing	5	9548	T-Beam	Unbalanced widening section support at diaphragms until completion of closure pour.
B-N O'Xing	5	9566	Box Girder	Widened with P.S. Girders, X-beams, and diaphragms not in line with existing jacking required to manipulate stresses, added enclusure walls.
Blakeslee Jct. E/W	5	9638	T-Beam and Box Girder	Post-tensioned X-beam, single web.
B-N O'Xing	18	9688	Box Girder	
SR 536		9696	T-Beam	Similar to Contract 9548.
LE Line over Yakima River	90	9806	Box Girder	Pier shaft.
SR 18 O-Xing	90	9823	P.S. Girder	Lightweight concrete.
Hamilton Road O-Xing	5	9894	T-Beam	Precast girder in one span.
Dillenbauch Creek	5		Flat Slab	
Longview Wye SR 432 U-Xing	5		P.S. Girder	Bridge lengthening.
Klickitat River Bridge	142		P.S. Girder	Bridge replacement.
Skagit River Bridge	5		Steel Truss	Rail modification.
B-N O-Xing at Chehalis	5			Replacement of thru steel girder span with stringer span.
Bellevue Access EBCD Widening and Pier 16 Modification	90	3846	Flat Slab and Box Girder	Deep, soft soil. Stradle best replacing single column.
Totem Lake/NE 124th I/C	405	3716	T-Beam	Skew = 55 degrees.
Pacific Avenue I/C	5	3087	Box Girder	Complex parallel skewed structures.
SR 705/SR 5 SB Added Lane	5	3345	Box Girder	Multiple widen structures.
Mercer Slough Bridge 90/43S		3846	CIP Conc. Flat Slab	Tapered widening of flat slab outrigger pier, combined footings.
Spring Street O-Xing No. 5/545SCD		3845	CIP Conc. Box Girder	Tapered widening of box girder with hingers, shafts.
Fishtrap Creek Bridge 546/8		3661	P.C. Units	Widening of existing P.C. Units. Tight constraints on substructure.
Columbia Drive O-Xing 395/16		3379	Steel Girder	Widening/Deck replacement using standard rolled sections.

Bridge	SR	Contract No.	Type of Bridge	Unusual Features
S 74th-72nd St. O-Xing No. 5/426		3207	CIP Haunched Con. Box Girder	Haunched P.C. P.T. Bath Tub girder sections.
Pacific Avenue O-Xing No. 5/332		3087	CIP Conc. Box Girder	Longitudinal joint between new and existing.
Tye River Bridges 2/126 and 2/127		3565	CIP Conc. Tee Beam	Stage construction with crown shift.
SR 20 and BNRR O-Xing No. 5/714		9220	CIP Conc. Tee Beam	Widened with prestressed girders raised crossbeam.
NE 8th St. U'Xing No. 405/43		9267	Prestressed Girders	Pier replacement — widening.
So. 212th St. U'Xing SR 167		3967	Prestressed Girders	Widening constructed as stand alone structure. Widening column designed as strong column for retrofit.
SE 232nd St. SR 18		5801	CIP Conc. Post-tensioned Box	Skew = 50 degree. Longitudinal "link pin" deck joint between new and existing to accommodate new creep.
Obdashian Bridge 2/275		N/A 1999	CIP Post-tensioned Box	Sidewalk widening with pipe struts.

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
8759	Kalama River Bridge SB	Cowlitz	2/70	40 200 200 40	Varies 46-53		0	6′ sidewalk on one side.
	NB		2/70	40 200 200 40	Varies 46.5	Varies	0	6′ sidewalk on one side.
8761	Valley View Road O'xing	Snohomish	2/70	88 170 88	38	25.2	0	
9102	Columbia River Bridge at Olds**	Chelan & Douglas	7/71	190 260 190	74	Varies	0	
9749	Evergreen Parkway	Thurston		100.5 145 145 114 114 87.5	26	Varies	47	Hourglass columns.
9840	W Sunset Way Ramp U'xing	King	12/74	160 159 100	26	22.9	Curved 500'R & 600'R	
1193	24F Over MD Line	Clark	8/78	129 201 129	26	Varies	0	
3794	Sen. Sam C. Guess Memorial (Division St. 2/644)		5/90	126 182 126	77	Varies	12	Replaced arch, built in two stages.

**Middle 3 spans of 7-span bridge are post-tensioned.

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
8569	Brickyard Road U'xing	King	2/69	137 155	38	22.2	45	
9122	NE 50th Avenue U'xing	Clark	7/71	124 124	44	24.8	12	
9122	NE 69th Avenue U'xing	Clark	7/71	130 130	84	23.6	0	
9289	SE 232nd Street U'xing	King	3/72	141 133	55	23.5	51	
9448	NE 18th Street U'xing	Clark	1/73	138 138	44	22.8	17	6′ sidewalk on each side.
9737	Mill Plain Road I/C U'xing	Clark	5/74	167 172	84	22.2	8	5′ sidewalk on each side.
0862	East Zillah I/C U'xing	Yakima	10/77	178 158	40	23.0	44	
0862	Hudson Road U'xing	Yakima	10/77	151 151	30	22.6	37	
1219	Johnson Road U'xing	Yakima & Benton	8/78	156 161	34	22.7	45	
1366	Donald Road U'xing	Yakima	12/78	142 155	55	23.8	45	
1764	148th Avenue NE U'xing	King	12/79	168 157	60	21.9	41	
1788	Gap Road U'xing	Yakima	1/80	131 131	30	22.1	37	

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
2156	14-H Line U'xing	Clark	11/81	114 114	60	22.8	0	
2156	14-D Line U'xing (North)	Clark	11/81	196 196	26	21.8	Curbed 600'R	
2217	SR 12 U'xing	Benton	2/82	147 147	55	23.3	0	
2217	Keene Road U'xing	Benton	2/82	150 150	34	21.4	Curved 11,459'R	25' counterweighted cantilever spans at each end. Transv. P.T.
2207	G Line U'xing	Benton	4/82	162.4 180.6	Varies 78.6-84.6	20.5	0	30' counterweighted cantilever spans at each end. Transv. P.T.
2207	N-S Line U'xing	Benton	4/82	155 155	38	22.1	0	
2207	SR 240 Connection U'xing (R-Line)	Benton	4/82	163.5 163.5	72	20.4	0	25' counterweighted cantilever spans at each end. Transv. P.T.
2236	Road 68 I/C U'xing	Franklin	4/82	191 191	64	23.2	35	
2236	Road 100 I/C U'xing	Franklin	4/82	183 167	55	21.5	15	
2236	SR 14 I/C U'xing (Eastbound)	Franklin	4/82	170 156	26	22.4	Curved 1600'R	
2236	SR 14 I/C U'xing (Westbound)	Franklin	4/82	159 148	38	21.8	Curved 1500'R	

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
8759	Kalama River Bridge SB	Cowlitz	2/70	40 200 200 40	Varies 46-53		0	6′ sidewalk on one side.
	NB		2/70	40 200 200 40	Varies 46.5	Varies	0	6′ sidewalk on one side.
8761	Valley View Road O'xing	Snohomish	2/70	88 170 88	38	25.2	0	
9102	Columbia River Bridge at Olds**	Chelan & Douglas	7/71	190 260 190	74	Varies	0	
9749	Evergreen Parkway	Thurston		100.5 145 145 114 114 87.5	26	Varies	47	Hourglass columns.
9840	W Sunset Way Ramp U'xing	King	12/74	160 159 100	26	22.9	Curved 500'R & 600'R	
1193	24F Over MD Line	Clark	8/78	129 201 129	26	Varies	0	
3794	Sen. Sam C. Guess Memorial (Division St. 2/644)		5/90	126 182 126	77	Varies	12	Replaced arch, built in two stages.

**Middle 3 spans of 7-span bridge are post-tensioned.

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
1439	SR 516 O'xing	King	3/79	63.5 133 63.5	42	24.2	40	
1580	Ahtanum Creek O'xing SB	Yakima	8/79	167 5@172 167	26	25.1	Curved 1200'R	
	NB		8/79	137 6@172 166	38	25.1	Curved 1200'R	
1950	Yakima River Bridges North Bridge	Benton	10/80	140+ 161 161 215 147	Varies 48'-100'	Varies	Curved 6000'R	Transverse post-tentioning.
	South Bridge			140+ 161 161 215 147	38	Varies	Curved 5900'R	Transverse post-tensioning. 10' bicycle and pedestrian path on one side.
2156	14-I Line	Clark	11/81	163 145 82	38	22.2	Curved 600'R	
2156	14 D Line (South)	Clark	11/81	128 171 128	26	24.4	Curved 625'R	
2207	GE Line Over G Line	Benton	4/82	90 188 90	38	23.5	Curved 1400'R	

Contract No.	Name	County	Award Date	Span	Width Curb Curb (ft.)	Span/ Depth	Skew Deg.	Remarks
2207	RA Line Over ER Line	Benton	4/82	47 104 47	55	17.3	20	Transverse post-tentioning.
2245	Pearl Street O'xing	Pierce	4/82	49 159 49	54	22.7	Curved 1400'R	
2245	6th Avenue O'xing	Pierce	4/82	43 125 43	Varies 87.4-102	22.7	Curved 1400'R & 400'R	
2327	Spokane River Bridge Stage 1	Spokane	6/82	175 255 175	76	Varies	0	Transverse post-tentioning.
***	Green River Bridge	King		118 150 99	74	Varies	22	
3794	Sen. Sam C. Guess Memorial (Division St. 2/644)		5/90	126 182 126	77	Varies (depth 5.5 to 8.5 at piers)	12	Replaced arch, built in two stages.

***Not yet to contract.

Design Example 1 Prestressed Girder Design

Prestressed Girder Design (LRFD)

girder := "interior"

Design Criteria

Loading: HL-93

Concrete:

prestressed girder, $f'_{ci} := 7.5 \cdot ksi$

$f'_{c} := 8.5 \cdot ksi$ (f'ci + 1 ksi)

slab, $f'_{cs} := 4 \cdot ksi$

Reinforcing Steel: (§5.4.3)
AASHTO M-31, Grade 60, $f_y := 60 \cdot ksi$

$E_s := 29000 \cdot ksi$

Prestressing Steel:
AASHTO M-203, uncoated 0.6"ϕ, 7 wire, low-relaxation strands (§5.4.4.1)

$f_{pu} := 270 \cdot ksi$

$f_{py} := 0.90 \cdot f_{pu}$ $f_{py} = 243\,ksi$

$E_p := 28500 \cdot ksi$

Nominal strand diameter, $d_b := 0.6 \cdot in$

$A_p := 0.217 \cdot in^2$

Design Method: LRFD

1. Structure

Simple span design

Bridge width, $BW := 38 \cdot ft$ curb to curb

Girder spacing, $S := 8 \cdot ft$

Number of girder lines, $N_b := 5$

Skew angle, $\theta_{sk} := 40.37 \cdot deg$

Design span, CL of brg. to CL of brg., $L := 131.75 \cdot ft$

Girder length (see girder schedule), $GL := 133.07 \cdot ft$

Distance from end of girder to CL Brg. (see girder schedule), $P2 := \dfrac{3 \cdot in}{\cos(\theta_{sk})}$

Curb width on deck, $cw := 10.5 \cdot in$

Deck overhang (from CL of exterior girder to end of deck, see slab design),

$$overhang := \frac{BW - (N_b - 1) \cdot S}{2} + cw \qquad overhang = 3.875\,ft$$

Prestressing,

harping strands $N_h := 16$

straight strands $N_s := 26$

temporary strands $N_t := 2$

2. Live Load

Loading : HL-93, consisted of a combination of the (§3.6.1.2.1)

- Design truck (HS20) or design tandem (2-25 kip axles @ 4'-0" apart), and
- Design lane load (no dynamic load allowance) $w_{lane} := 0.64 \cdot \dfrac{kip}{ft}$

No. of design lanes (§3.6.1.1.1)

$$N_L := \begin{cases} floor\left(\dfrac{BW}{12 \cdot ft}\right) & \text{if } BW > 24 \cdot ft \\ 2 & \text{if } 24 \cdot ft \geq BW \geq 20 \cdot ft \\ 1 & \text{otherwise} \end{cases} \qquad N_L = 3$$

3. Concrete Properties

3.1 Precast Prestressed Girder

$$w_c := 0.160 \cdot kcf$$

$$E_c := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \qquad E_c = 6157\,ksi \qquad (§5.4.2.4)$$

$$E_{ci} := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi \qquad E_{ci} = 5784\,ksi$$

$$f_r := 0.24 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \qquad f_r = 0.7\,ksi \qquad (\S 5.4.2.6)$$

4 Concrete Deck Slab Requirement

4.1 Determination of slab thickness and effective width

use slab depth (see slab design)

for design, $t_{s1} := 7.0 \cdot in$

for D.L. calculation, $t_{s2} := 7.5 \cdot in$

5. Computation of Section Properties

5.1 Stiffness Assumptions
Dead loads to non-composite section.
Live load and S.I.D.L. to composite section.

5.2 Prestressed Girder Section Properties
W74G Washington standard prestressed girder

$d_g := 73.5 \cdot in$ $\quad A_g := 747.7 \cdot in^2$ $\quad I_g := 547384 \cdot in^4$ $\quad Y_{bg} := 38.02 \cdot in$ $\quad b_w := 6 \cdot in$

$Y_{tg} := d_g - Y_{bg}$ $\quad S_{tg} := \frac{I_g}{Y_{tg}}$ $\quad S_{bg} := \frac{I_g}{Y_{bg}}$ $\quad b_f := 43 \cdot in$ (top flange width)

$Y_{tg} = 35.48\,in$ $\quad S_{tg} = 15428\,in^3$ $\quad S_{bg} = 14397\,in^3$

"A" dimension (from top of slab to top of girder) (BDM 6.1-A1-1), set $A := 11.25 \cdot in$

Optional criteria for span-to depth ratio (§2.5.2.6.3) - for continuous prestressed girder, the min. depth (including deck) is $0.040 \cdot L = 5.27\,ft$ **OK**

5.3 Composite Section Properties (§4.6.2.6)

Assume effective span (for simple span analysis), $L_e := L$

Let $tem := \max\left(\left(b_w \quad \frac{1}{2} \cdot b_f\right)\right)$

The effective flange width b shall be taken as the least of

$$b_i := \min\left(\begin{pmatrix} \frac{1}{4}\cdot L_e \\ 12\cdot t_{s1} + tem \\ S \end{pmatrix}\right) \qquad b_i = 96\,in \qquad b := \begin{vmatrix} b_i \text{ if girder = "interior"} \\ \frac{b_i}{2} + \min\left(\begin{pmatrix} \frac{1}{8}\cdot L_e \\ 6\cdot t_{s1} + \frac{tem}{2} \\ overhang \end{pmatrix}\right) \text{ if girder = "exterior"} \end{vmatrix}$$

$$b = 8\,ft$$

modular ratio, $n := \sqrt{\frac{f'_c}{f'_{cs}}}$ $\qquad n = 1.458$

$$A_{slab} := \frac{b}{n}\cdot t_{s1} \qquad Y_{bs} := d_g + \frac{t_{s1}}{2} \qquad AY_{bs} := A_{slab}\cdot Y_{bs}$$

	Area	Y_b	$A\cdot Y_b$
slab	$A_{slab} = 461\,in^2$	$Y_{bs} = 77\,in$	$A_{slab}\cdot Y_{bs} = 35496\,in^3$
girder	$A_g = 747.7\,in^2$	$Y_{bg} = 38.02\,in$	$A_g\cdot Y_{bg} = 28428\,in^3$

$Y_b := \frac{A_{slab}\cdot Y_{bs} + A_g\cdot Y_{bg}}{A_{slab} + A_g}$ $\qquad Y_b = 52.89\,in$ $\qquad$ @ bottom of girder

$Y_t := d_g - Y_b$ $\qquad Y_t = 20.61\,in$ $\qquad$ @ top of girder

$Y_{ts} := t_{s1} + Y_t$ $\qquad Y_{ts} = 27.61\,in$ $\qquad$ @ top of slab

$$I_{slabc} := A_{slab}\cdot\left(Y_{ts} - \frac{t_{s1}}{2}\right)^2 + \frac{\left(\frac{b}{n}\right)\cdot t_{s1}^{\;3}}{12} \qquad I_{slabc} = 269922\,in^4$$

$$I_{gc} := A_g\cdot\left(Y_b - Y_{bg}\right)^2 + I_g \qquad I_{gc} = 712642\,in^4$$

$$I_c := I_{slabc} + I_{gc} \qquad I_c = 982564\,in^4$$

Section modulous of the composite section

$S_b := \frac{I_c}{Y_b}$ $\qquad S_b = 18579\,in^3$ $\qquad$ @ bottom of girder

$S_t := \frac{I_c}{Y_t}$ $\qquad S_t = 47667\,in^3$ $\qquad$ @ top of girder

$S_{ts} := \frac{n\cdot I_c}{Y_{ts}}$ $\qquad S_{ts} = 51871\,in^3$ $\qquad$ @ top of slab $\qquad$ (modified due to modular ratio)

6 Limit States

6.1 Service Limit States

Limit states relating to stress, deformation, and crack width under regular service conditions.

6.2 Load Factors and Combinations (§3.4.1)

Service I - Load combination relating to the normal operational use of the bridge. Compression in prestressed components is investigated using this load combination.

1.0 DC + 1.0 (LL+IM)

Service III - Load combination relating only to tension in prestressed concrete structures with the objective of crack control.

1.0 DC + 0.8 (LL+IM)

Force effects due to temperature, shrinkage and creep, because of the free movement at end piers, are considered to be zero.

Force effects due to temperature gradient, wind, friction at bearings, and settlement are ignored.

7 Vehicular Live Load

7.1 Design Live Load (§3.6.1.2.2)

AASHTO HS20-44

$M_{hs20}(L) = 2094\,kip \cdot ft$ per lane

Tandem does not control.

AASHTO Lane Load

$M_{lane} := w_{lane} \cdot \frac{L^2}{8}$ $M_{lane} = 1389\,kip \cdot ft$

7.2 Dynamic Load Allowance, IM (§3.6.2)

The dynamic load allowance shall not applied to pedestrian loads or to the design lane load.

All other components (including girder)

Fatigue and Fracture limit state IM = 15%
All other limit states

$IM := 33 \cdot \%$

$M_{LL} := M_{hs20}(L) \cdot (1 + IM) + M_{lane}$ $M_{LL} = 4174\,kip \cdot ft$

7.3 Distribution of Live Load

7.3.1 D.F. for Moment (interior girder)

Range of applicability (LRFD Table 4.6.2.2.2b-1), case k

Width of deck is constant (§4.6.2.2.1)

Curvature in plan is less than the limit specified in §4.6.1.2

$\text{if}(3.5 \cdot ft \le S \le 16.0 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(4.5 \cdot in \le t_{s1} \le 12.0 \cdot in, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(20 \cdot ft \le L \le 240 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(N_b \ge 4, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

The multiple presence factors shall not be applied except for exterior girders where special requirement applied (§3.6.1.1.2 & 4.6.2.2.2d).

e_g, distance between the centers of gravity of the basic beam and deck,

$e_g := Y_{bs} - Y_{bg}$ $\qquad$ $e_g = 39\,in$

$K_g := n \cdot \left(I_g + A_g \cdot e_g^{\,2}\right)$ $\qquad$ $K_g = 2.45 \times 10^6\,in^4$ $\qquad$ (LRFD Eq. 4.6.2.2.1-1)

$\text{if}\left(10^4 in^4 \le K_g \le 7 \cdot 10^6 in^4, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

D.F. for moment (interior girder, two or more design lanes loaded governs by inspection):

$$DF_i := 0.075 + \left(\frac{S}{9.5 \cdot ft}\right)^{0.6} \cdot \left(\frac{S}{L}\right)^{0.2} \cdot \left(\frac{K_g}{L \cdot t_{s1}^{\,3}}\right)^{0.1} \qquad DF_i = 0.674$$

7.3.2 D.F. for moment (exterior girder)

Range of applicability (LRFD Table 4.6.2.2.2d-1), case k

$\text{if}(-1.0 \cdot ft \le overhang - cw \le 5.5 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ $\qquad$ $overhang - cw = 3\,ft$

$\text{if}(N_b > 3, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$$ee := \max\left(\begin{pmatrix} 0.77 + \dfrac{overhang - cw}{9.1 \cdot ft} \\ 1.0 \end{pmatrix}\right) \qquad ee = 1.1$$

D.F. for moment (exterior girder)

$DF_e := ee \cdot DF_i$ $\qquad$ $DF_e = 0.741$ $\qquad$ lever rule check (§4.6.2.2.2d) (see hand calculation) $\qquad$ set $\qquad$ $DF_e := DF_e$

7.3.3 Skewed Bridges (§4.6.2.2.2e)

Applied when the difference between skew angles of two adjacent lines of supports does not exceed 10 deg.

$\text{if}(30 \cdot deg \le \theta_{sk} \le 60 \cdot deg, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(3.5 \cdot ft \le S \le 16.0 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(20 \cdot ft \le L \le 240 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(N_b \ge 4, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$$c_1 := \begin{cases} 0.0 & \text{if } \theta_{sk} < 30 \cdot deg \\ 0.25 \cdot \left(\dfrac{K_g}{L \cdot t_{s1}{}^3}\right)^{0.25} \cdot \left(\dfrac{S}{L}\right)^{0.5} & \text{otherwise} \end{cases}$$

$c_1 = 0.09$

$$SK := \begin{cases} 1 - c_1 \cdot (\tan(60 \cdot deg))^{1.5} & \text{if } \theta_{sk} > 60 \cdot deg \\ 1 - c_1 \cdot (\tan(\theta_{sk}))^{1.5} & \text{otherwise} \end{cases}$$

$SK = 0.93$

Reduced D.F. for moment

$$DF := \begin{cases} SK \cdot DF_i & \text{if girder} = \text{"interior"} \\ (SK \cdot DF_e) & \text{if girder} = \text{"exterior"} \end{cases}$$

$DF = 0.627$

8 Computation of Stresses

8.1 Stresses due to Weight of Girder

$w_g := A_g \cdot w_c$ $\qquad w_g = 0.831 \dfrac{kip}{ft}$

$M_g := w_g \cdot \dfrac{L^2}{8}$ $\qquad M_g = 1803\, kip \cdot ft$

$f_{tg} := \dfrac{-M_g}{S_{tg}}$ $\qquad f_{tg} = -1.4\, ksi$ $\qquad$ @ top of girder

$f_{bg} := \dfrac{M_g}{S_{bg}}$ $\qquad f_{bg} = 1.5\, ksi$ $\qquad$ @ bottom of girder

8.2 Stress due to Weight of Slab and Pad

$$w_s := \begin{cases} t_{s2} \cdot (S) \cdot w_c & \text{if girder} = \text{"interior"} \\ \left[t_{s2} \cdot \left(\dfrac{S}{2} + \text{overhang}\right) \cdot w_c\right] & \text{if girder} = \text{"exterior"} \end{cases}$$

$w_s = 0.8 \dfrac{kip}{ft}$

The depth of slab pad is fillet depth at the center line of span,

$w_{pu} := (A - t_{s2}) \cdot b_f \cdot w_c$ $\qquad w_{pu} = 0.18 \dfrac{kip}{ft}$ $\qquad$ assume uniform distribution

$w_{spu} := w_s + w_{pu}$ $\qquad w_{spu} = 0.98 \dfrac{kip}{ft}$

$M_{sp}(x) := w_{spu} \cdot \dfrac{L}{2} \cdot x - w_{spu} \cdot \dfrac{x^2}{2}$ $\qquad M_{sp}\left(\dfrac{L}{2}\right) = 2125\, kip \cdot ft$

$$f_{ts} := \frac{-M_{sp}\left(\frac{L}{2}\right)}{S_{tg}} \qquad f_{ts} = -1.65\,ksi \qquad \text{@ at top of girder}$$

$$f_{bs} := \frac{M_{sp}\left(\frac{L}{2}\right)}{S_{bg}} \qquad f_{bs} = 1.77\,ksi \qquad \text{@ bottom of girder}$$

8.3 Stresses due to Weight of Diaphragm

Intermediate diaphragms are not required (§5.13.2.2) for straight girders; but BDM 6.2-A5 requires intermediate diaphragms @ **1/4 point of span for span over 120 ft.**

Number of diaphragms, $n_{diaph} := 3$

Diaphragm section (BDM 6.3-A1), $intd := 3.625ft \cdot 8 \cdot in$

$$P := \begin{cases} intd \cdot \left(\frac{S - b_w}{\cos(\theta_{sk})}\right) \cdot w_c & \text{if girder = "interior"} \\ \frac{intd}{2} \cdot \left(\frac{S - b_w}{\cos(\theta_{sk})}\right) \cdot w_c & \text{if girder = "exterior"} \end{cases} \qquad P = 3.8\,kip$$

$$M_d := \begin{cases} P \cdot [L - (0.3 + 0.1)L] & \text{if } n_{diaph} = 4 \\ P \cdot \left(\frac{1.5 \cdot L}{2} - \frac{L}{4}\right) & \text{if } n_{diaph} = 3 \\ P \cdot \left(\frac{L}{2} - \frac{L}{6}\right) & \text{if } n_{diaph} = 2 \\ \frac{P}{2} \cdot \left(\frac{L}{2}\right) & \text{if } n_{diaph} = 1 \\ 0 \cdot kip \cdot ft & \text{otherwise} \end{cases} \qquad M_d = 250.7\,kip \cdot ft$$

$$f_{td} := \frac{-M_d}{S_{tg}} \qquad f_{td} = -0.2\,ksi \qquad \text{@ top of girder}$$

$$f_{bd} := \frac{M_d}{S_{bg}} \qquad f_{bd} = 0.21\,ksi \qquad \text{@ bottom of girder}$$

8.4 Concrete Stresses due to S.I.D.L. (Applied to Composite Section)

Weight of one traffic barrier is $tb := 0.47 \cdot \frac{kip}{ft}$

Weight of one traffic barrier is distributed over min. of $\frac{N_b}{2} = 2.5$ girders or 3 girders.

$$w_b := \frac{tb}{\min((N_b \cdot 0.5 \;\; 3))} \qquad w_b = 0.188 \frac{kip}{ft}$$

$$M_b := w_b \cdot \frac{L^2}{8} \qquad M_b = 407.9\,kip \cdot ft$$

$$f_{tsb} := \frac{-M_b}{S_{ts}} \qquad f_{tsb} = -0.09\,ksi$$ @ top of slab

$$f_{tb} := \frac{-M_b}{S_t} \qquad f_{tb} = -0.1\,ksi$$ @ top of girder

$$f_{bb} := \frac{M_b}{S_b} \qquad f_{bb} = 0.26\,ksi$$ @ bottom of girder

8.5 Concrete Stress due to Live Load (Applied to Composite Section)

$$M_L := M_{LL} \cdot DF \qquad M_L = 2615\,kip \cdot ft$$

Service I

$$f_{tsL} := \frac{-M_L}{S_{ts}} \qquad f_{tsL} = -0.61\,ksi$$ @ top of slab

$$f_{tL} := \frac{-M_L}{S_t} \qquad f_{tL} = -0.66\,ksi$$ @ top of girder

Service III

$$f_{bL} := \frac{0.8 \cdot M_L}{S_b} \qquad f_{bL} = 1.35\,ksi$$ @ bottom of girder

8.6 Summary of Stresses at Mid-span

	@ top of slab	*@ top of girder*	*@ bot. of girder*
girder		$f_{tg} = -1.4\,ksi$	$f_{bg} = 1.5\,ksi$
slab+pad		$f_{ts} = -1.65\,ksi$	$f_{bs} = 1.77\,ksi$
diaphragm		$f_{td} = -0.2\,ksi$	$f_{bd} = 0.21\,ksi$
s.i.d.l.	$f_{tsb} = -0.09\,ksi$	$f_{tb} = -0.1\,ksi$	$f_{bb} = 0.26\,ksi$
LL+IM (Service I)	$f_{tsL} = -0.61\,ksi$	$f_{tL} = -0.66\,ksi$	

LL+IM (Service III) $f_{bL} = 1.35\,ksi$

Service I - 1.0 DC +1.0 (LL+IM)

$f_{tgI} := f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tL}$ $f_{tgI} = -4.01\,ksi$

Service III - 1.0 DC +0.8 (LL+IM)

$f_{bgIII} := f_{bg} + f_{bs} + f_{bd} + f_{bb} + f_{bL}$ $f_{bgIII} = 5.1\,ksi$

9 Determination of Prestressing Forces (§5.9)

9.1 Stress Limits for Prestressing Strands

$f_{pu} = 270\,ksi$ (LRFD Table 5.4.4.1-1)

$f_{py} = 243\,ksi$

$f_{pe} := 0.80 \cdot f_{py}$ $f_{pe} = 194.4\,ksi$ @ service limit state after all losses (LRFD Table 5.9.3-1)

Losses due to steel relaxation at transfer (§5.9.5.4.4b)

Curing time for concrete to attain f'ci is approximately 12 hours: set $t := 1.0$ day

$f_{pj} := 0.75 \cdot f_{pu}$ $f_{pj} = 202.5\,ksi$ immediately prior to transfer+steel relax. (LRFD Table 5.9.3-1)

$$\Delta f_{pR1} := \frac{\log(24.0 \cdot t)}{40.0} \cdot \left(\frac{f_{pj}}{f_{py}} - 0.55 \right) \cdot f_{pj} \qquad \Delta f_{pR1} = 1.98\,ksi$$

9.2 Allowable Concrete Stresses at Service Limit State

9.2.1 Compressive Stresses Limits After All Losses (§5.9.4.2.1)

Using Service I load combination

$-0.45 \cdot f'_c = -3.83\,ksi$ due to permanent loads

$-0.60 \cdot f'_c = -5.1\,ksi$ due to permanent and transient loads and during shipping and handling

$-0.40 \cdot f'_c = -3.4\,ksi$ due to live loads plus one-half the sum of effective prestress and permanent loads

9.2.2 Tensile Stress Limits (§5.9.4.2.2)

For the service load combinations which involves traffic loading, tension stress in members with bonded prestressing strands should be investigated using Service III load combination.

Tension in precompressed tensile zone assuming uncracked section

$0.190 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 0.55\,ksi$ OR $0.0948 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 0.28\,ksi$ (for severe corrosive conditions)

$0 \cdot ksi$ WSDOT design practice

9.3 Loss of Prestress (§5.9.5)

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

Given: $A_p = 0.217\,in^2$

harping strands	$N_h = 16$	jacking force,	$f_{pj} \cdot N_h \cdot A_p = 703.08\,kip$
straight strands	$N_s = 26$	jacking force,	$f_{pj} \cdot N_s \cdot A_p = 1143\,kip$
temporary strands	$N_t = 2$	jacking force,	$f_{pj} \cdot N_t \cdot A_p = 87.88\,kip$

(note: these forces include initial prestress relaxation loss, see §C5.9.5.4.4b)

$N_p := N_s + N_h$ $\quad N_p = 42$

$A_{ps} := A_p \cdot N_p$ $\quad A_{ps} = 9.114\,in^2$

$A_{temp} := A_p \cdot N_t$ $\quad A_{temp} = 0.434\,in^2$

$A_{pstemp} := A_p \cdot (N_t + N_p)$ $\quad A_{pstemp} = 9.548\,in^2$

Find E,

$$E := \begin{cases} 2 \cdot in & \text{if } N_s \le 10 \\ 2 \cdot in + \dfrac{(N_s - 10) \cdot (2 \cdot in)}{N_s} & \text{if } 10 < N_s \le 18 \\ 2 \cdot in + \dfrac{8 \cdot (2 \cdot in) + (N_s - 18) \cdot (4 \cdot in)}{N_s} & \text{if } 18 < N_s \le 22 \\ 2 \cdot in + \dfrac{(N_s - 14) \cdot (2 \cdot in) + 4 \cdot (4 \cdot in)}{N_s} & \text{if } 22 < N_s \le 24 \\ 2 \cdot in + \dfrac{10 \cdot (2 \cdot in) + (N_s - 20) \cdot (4 \cdot in)}{N_s} & \text{otherwise} \end{cases}$$

$E = 3.69\,in$

Note: Don't use this E value for W42G girders (see std. plans).

$$F_{CL} := \text{if}\left[N_h > 12, \frac{(N_h - 12)\cdot(3\cdot in)}{N_h} + 3\cdot in, 3\cdot in\right] \qquad F_{CL} = 3.75\,in$$

c.g. of straight strands to c.g. of girder, $e_s := Y_{bg} - E$ $\quad e_s = 34.33\,in$

c.g. of harped strands to c.g. of girder, $e_h := Y_{bg} - F_{CL}$ $\quad e_h = 34.27\,in$

c.g. of temporary strands to c.g. of girder, $e_{temp} := Y_{tg} - 2\cdot in$ $\quad e_{temp} = 33.48\,in$

c.g. of all strands to c.g. of girder, $e_p := \dfrac{e_s \cdot N_s + e_h \cdot N_h}{N_p}$ $\quad e_p = 34.31\,in$

$$e_{ptemp} := \frac{e_p \cdot N_p - e_{temp} \cdot N_t}{N_p + N_t} \qquad e_{ptemp} = 31.22\,in$$

9.3.1 Loss due to Elastic Shortening, Δf_{pES} (§5.9.5.2.3a)

$f_{cgp:}$ concrete stress at c.g. of prestressing tendons due to the prestressing force at transfer and the self-weight of the member at the sections of maximum moment.

Guess values: $p_{st} := 180\cdot ksi$ prestress tendon stress at transfer (LRFD Table 5.9.3-1)

Given $$(f_{pj} - \Delta f_{pR1} - p_{st})\cdot\frac{E_{ci}}{E_p} = -\left[\frac{-(p_{st}\cdot A_{pstemp})}{A_g} - \left[\frac{(p_{st}\cdot A_{pstemp})\cdot e_{ptemp}^{\,2}}{I_g}\right] + \frac{w_g\cdot(GL)^2}{8}\cdot\frac{e_{ptemp}}{I_g}\right]$$

$p_{st} := \text{Find}(p_{st})$ $\quad p_{st} = 180.3\,ksi$

$$f_{cgp} := \frac{-[p_{st}\cdot(A_{pstemp})]}{A_g} - \left[\frac{(p_{st}\cdot A_{pstemp})\cdot e_{ptemp}^{\,2}}{I_g}\right] + \frac{w_g\cdot(GL)^2}{8}\cdot\frac{e_{ptemp}}{I_g} \qquad f_{cgp} = -4.11\,ksi$$

$\Delta f_{pES} := f_{pj} - \Delta f_{pR1} - p_{st}$ $\quad \Delta f_{pES} = 20.25\,ksi$

9.3.2 Approximate Lump Sum Estimate of Time Dependent Losses (§5.9.5.3)

Time-dependent losses : $\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

Criteria: $f'_{ci} > 3.5\cdot ksi = 1$ **OK**

Normal density concrete
Concrete is either steam or moist cured

Prestressing is by low relaxation strands
Are sited in average exposure condition and temperatures

no partial prestressing $A_s := 0 \cdot in^2$

partial prestress ratio (LRFD Eq. 5.5.4.2.1-2) $PPR := \dfrac{A_{ps} \cdot f_{py}}{A_{ps} \cdot f_{py} + A_s \cdot f_y}$ $PPR = 1$

Approximate lump sum estimate of time-dependent losses (§5.9.5.3)

$$LOSS_t := 33.0 \cdot \left(1.0 \cdot ksi - 0.15 \cdot \frac{f'_c - 6.0 \cdot ksi}{6.0}\right) + 6.0 \cdot ksi \cdot PPR \qquad LOSS_t = 36.94\,ksi$$

Allowable reduction for I-girders, 6.0 ksi,

$$LOSS_t := LOSS_t - 6.0 \cdot ksi \qquad LOSS_t = 30.9\,ksi$$

(§5.9.5.1) In pretension members where the approximate lump sum estimate of losses is used, Δf_{pR1} should be deducted from the total relaxation.
At transfer, the losses that could be accounted for are elastic shortening and steel relaxation only.

$$LOSS_t := LOSS_t - \Delta f_{pRl} \qquad LOSS_t = 28.96\,ksi$$

9.3.3 Loss due to Creep Δf_{pCR} (§5.9.5.2.3a)

Δf_{cdp}, change in concrete stress at center of gravity of prestressing steel due to permanent loads, except the load acting at the time the prestressing force is applied. Values of Δf_{cdp} should be calculated at the same section or sections for which f_{cgp} is calculated.

c.g. of straight strands to c.g. of composite girder, $e_{sc} := Y_b - E$

$e_{sc} = 49.19\,in$

c.g. of harped strands to c.g. of composite girder, $e_{hc} := Y_b - F_{CL}$

$e_{hc} = 49.14\,in$

c.g. of all strands to c.g. of composite girder, $e_{pc} := \dfrac{e_{sc} \cdot N_s + e_{hc} \cdot N_h}{N_p}$

$e_{pc} = 49.17\,in$

$$\Delta f_{cdp} := \frac{\left(M_{sp}\left(\frac{L}{2}\right) + M_d\right) \cdot e_p}{I_g} + \frac{M_b \cdot e_{pc}}{I_c} \qquad \Delta f_{cdp} = 2.03\,ksi$$

$$\Delta f_{pCR} := 12.0 \cdot (-f_{cgp}) - 7.0 \cdot \Delta f_{cdp} \qquad \Delta f_{pCR} = 35.09\,ksi$$ (note: > LOSSt sometimes)

Total loss Δf_{pT} (note: BDM assumes a 48 ksi total loss), not including Δf_{pRl},

$\Delta f_{pT} := LOSS_t + \Delta f_{pES}$ $\Delta f_{pT} = 49.21\,ksi$ **use** $\Delta f_{pT} := \Delta f_{pT}$

$f_{pe} := f_{pj} - \Delta f_{pRl} - \Delta f_{pT}$ $f_{pe} = 151.315\,ksi$ (use Modified Rate of Creep method)

$f_{pe} \le 0.80 \cdot f_{py} = 1$ OK, (LRFD Table 5.9.3-1)

$P_e := N_p \cdot A_p \cdot f_{pe}$ $P_e = 1379.1\,kip$

$$-\frac{P_e}{A_g} - P_e \cdot \frac{e_p}{S_{bg}} = -5.13\,ksi$$

$f_{bgIII} = 5.1\,ksi$

Tension at bottom of girder

$$f_{bgIII} + \left(-\frac{P_e}{A_g} - P_e \cdot \frac{e_p}{S_{bg}}\right) = -0.03\,ksi$$ < allowable $0.0948 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 0.28\,ksi$ **say OK**

$0 \cdot ksi$ (BDM)

10 Stresses at Service Limit State

10.1 Final Stresses at Midspan

Compressive stresses at top of slab

Stresses due to permanent load + prestressing

$f_{tsb} = -0.09\,ksi$ < allowable $-0.45 \cdot f'_{cs} = -1.8\,ksi$ **OK**

Stresses due to permanent and transient loads,

LL+IM (Service I) $f_{tsL} := \frac{-M_L}{S_{ts}}$ $f_{tsL} = -0.61\,ksi$

$f_{tsb} + f_{tsL} = -0.7\,ksi$ < allowable $-0.60 \cdot f'_{cs} = -2.4\,ksi$ **OK**

Stresses due to live load + one-half the sum of effective prestress and permanent loads,

$f_{tsL} + 0.5 \cdot f_{tsb} = -0.65\,ksi$ < allowable $-0.40 \cdot f'_{cs} = -1.6\,ksi$ **OK**

Stresses at top of girder

prestressing, $f_{tp} := -\frac{P_e}{A_g} + P_e \cdot \frac{e_p}{S_{tg}}$ $f_{tp} = 1.22\,ksi$

Stresses due to permanent loads,

$f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tp} = -2.13\,\text{ksi}$ < allowable $-0.45 \cdot f'_c = -3.83\,\text{ksi}$ **OK**

Stresses due to permanent and transient loads,

LL+IM (Service I) $f_{tL} := \dfrac{-M_L}{S_t}$ $f_{tL} = -0.66\,\text{ksi}$

$f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tL} + f_{tp} = -2.79\,\text{ksi}$ < allowable $-0.60 \cdot f'_c = -5.1\,\text{ksi}$ **OK**

Stresses due to live load + one-half the sum of effective prestress and permanent loads,

$f_{tL} + 0.5 \cdot \left(f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tp}\right) = -1.72\,\text{ksi}$ < allowable $-0.40 \cdot f'_c = -3.4\,\text{ksi}$ **OK**

10.2 Final Stresses at Harping Point

Harping location from CL of brg., $x_h := 0.4 \cdot GL - P2$ $x_h = 52.9\,\text{ft}$

Stresses at top of slab

s.i.d.l., $M_{bh} := w_b \cdot \dfrac{L}{2} \cdot x_h - w_b \cdot \dfrac{x_h^2}{2}$ $M_{bh} = 392.09\,\text{kip} \cdot \text{ft}$

$f_{tsb} := \dfrac{-M_{bh}}{S_{ts}}$ $f_{tsb} = -0.09\,\text{ksi}$

LL+IM (Service I)

$M_{LL} := 4025 \cdot \text{kip} \cdot \text{ft}$ (see QConBridge output)

$M_{Lh} := M_{LL} \cdot DF$ $M_{Lh} = 2522\,\text{kip} \cdot \text{ft}$

$f_{tsL} := \dfrac{-M_{Lh}}{S_{ts}}$ $f_{tsL} = -0.58\,\text{ksi}$

Stresses due to permanent loads,

$f_{tsb} = -0.09\,\text{ksi}$ < allowable $-0.45 \cdot f'_{cs} = -1.8\,\text{ksi}$ **OK**

Stresses due to permanent and transient loads,

$f_{tsb} + f_{tsL} = -0.67\,\text{ksi}$ < allowable $-0.60 \cdot f'_{cs} = -2.4\,\text{ksi}$ **OK**

Stresses due to live load + one-half of the permanent loads,

$f_{tsL} + 0.5 \cdot f_{tsb} = -0.63\,\text{ksi}$ < allowable $-0.40 \cdot f'_{cs} = -1.6\,\text{ksi}$ **OK**

Compressive stresses at top of girder

girder, $M_{gh} := w_g \cdot \frac{L}{2} \cdot x_h - w_g \cdot \frac{x_h^2}{2}$ $M_{gh} = 1733\,\mathrm{kip \cdot ft}$

$$f_{tg} := \frac{-M_{gh}}{S_{tg}} \qquad f_{tg} = -1.35\,\mathrm{ksi}$$

slab+pad, $M_{sp}(x_h) = 2.042\,10^3 \cdot \mathrm{kip \cdot ft}$

$$f_{ts} := \frac{-M_{sp}(x_h)}{S_{tg}} \qquad f_{ts} = -1.59\,\mathrm{ksi}$$

diaphragm, $M_{dh} := \left| \begin{array}{l} P \cdot \left[1.5 \cdot x_h - \left(x_h - \frac{L}{4}\right)\right] \text{ if } n_{diaph} = 3 \\ P \cdot \left[x_h - \left(x_h - \frac{L}{3}\right)\right] \text{ if } n_{diaph} = 2 \\ \frac{P}{2} \cdot (x_h) \text{ if } n_{diaph} = 1 \\ 0 \cdot \mathrm{kip \cdot ft} \text{ otherwise} \end{array} \right.$ $M_{dh} = 226.1\,\mathrm{kip \cdot ft}$

$$f_{td} := \frac{-M_{dh}}{S_{tg}} \qquad f_{td} = -0.18\,\mathrm{ksi}$$

s.i.d.l., $f_{tb} := \frac{-M_{bh}}{S_t}$ $f_{tb} = -0.1\,\mathrm{ksi}$

prestressing, $f_{tp} := -\frac{P_e}{A_g} + P_e \cdot \frac{e_p}{S_{tg}}$ $f_{tp} = 1.22\,\mathrm{ksi}$

Stresses due to permanent load + prestressing,

$f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tp} = -1.99\,\mathrm{ksi}$ < allowable $-0.45 \cdot f'_c = -3.83\,\mathrm{ksi}$ **OK**

Stresses due to permanent and transient loads,

LL+IM (Service I) $f_{tL} := \frac{-M_{Lh}}{S_t}$ $f_{tL} = -0.63\,\mathrm{ksi}$

$f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tp} + f_{tL} = -2.62\,\mathrm{ksi}$ < allowable $-0.60 \cdot f'_c = -5.1\,\mathrm{ksi}$ **OK**

Stresses due to live load + one-half the sum of effective prestress and permanent loads,

$0.5 \cdot (f_{tg} + f_{ts} + f_{td} + f_{tb} + f_{tp}) + f_{tL} = -1.63\,ksi$ < allowable $-0.40 \cdot f'_c = -3.4\,ksi$ **OK**

Stresses at bottom of girder

girder, $f_{bg} := \dfrac{M_{gh}}{S_{bg}}$ $f_{bg} = 1.44\,ksi$

slab+pad, $f_{bs} := \dfrac{M_{sp}(x_h)}{S_{bg}}$ $f_{bs} = 1.7\,ksi$

diaphragm, $f_{bd} := \dfrac{M_{dh}}{S_{bg}}$ $f_{bd} = 0.19\,ksi$

s.i.d.l., $f_{bb} := \dfrac{M_{bh}}{S_b}$ $f_{bb} = 0.25\,ksi$

prestressing, $f_{bp} := \dfrac{-P_e}{A_g} - P_e \cdot \dfrac{e_p}{S_{bg}}$ $f_{bp} = -5.13\,ksi$

LL+IM (Service III) $f_{bL} := \dfrac{M_{Lh} \cdot 0.8}{S_b}$ $f_{bL} = 1.3\,ksi$

Stresses due to permanent and transient load + prestressing

Tension at bottom of girder

$f_{bg} + f_{bs} + f_{bd} + f_{bb} + f_{bL} + f_{bp} = -0.24\,ksi$ < allowable $0.0948 \cdot \sqrt{\dfrac{f'_c}{ksi}} \cdot ksi = 0.28\,ksi$ **OK**

$0 \cdot ksi$ (BDM)

11 Stresses at Transfer

(§5.8.2.3) The prestressing force may be assumed to vary linearly from zero at free end of strand to a maximum at the transfer length.

Transfer Length, D,

$D := 60 \cdot d_b$ $D = 36\,in$

11.1 Prestressing Losses at Transfer

Loss due to elastic shortening, (note: BDM assumes a 20 KSI loss at transfer),

$\Delta f_{pES} = 20.2\,ksi$

Prestress tendon stress at transfer (LRFD Table 5.9.3-1), p_{st},

$p_{st} = 180.27\,ksi$

11.2 Concrete Stress at Transfer at "D" from End of Girder

Assume the girder is supported at the both ends at transfer (per a girder fabricator),

Moment due to weight of girder,

$$M_{gD} := w_g \cdot \frac{GL}{2} \cdot D - \frac{w_g}{2} \cdot D^2 \qquad M_{gD} = 162.1\,kip \cdot ft$$

Prestressing force at transfer,

For straight strands, $P_{sis} := p_{st} \cdot N_s \cdot A_p$ $\quad P_{sis} = 1017\,kip$

For harped strands, $P_{sih} := p_{st} \cdot N_h \cdot A_p$ $\quad P_{sih} = 625.9\,kip$

For temporary strands, $P_{sit} := p_{st} \cdot N_t \cdot A_p$ $\quad P_{sit} = 78.238\,kip$

$$P_{si} := P_{sis} + P_{sih} + p_{st} \cdot N_t \cdot A_p \qquad P_{si} = 1721\,kip$$

Eccentricity for harped strand at "D" from end of girder, e_{Dh},

find distance from the top of girder to the c.g. of the harped strands at the end of girder, F_o,

$$row := floor\left(\frac{N_h}{2}\right) \qquad F_o := 4 \cdot in + \frac{\left[\sum_{i=0}^{row-1} (2 \cdot i \cdot 2 \cdot in)\right] + (N_h - 2 \cdot row) \cdot (row)\,2 \cdot in}{N_h}$$

$F_o = 11\,in$ $\qquad$ set $\quad F_o := 29.0 \cdot in$

harping rise, $R_h := d_g - F_o - F_{CL}$ $\quad R_h = 40.75\,in$

harped strand slope, $slope_h := \dfrac{R_h}{x_h + P2}$ $\quad slope_h = 0.064$

check harped strand slope, $if\left(\begin{vmatrix} slope_h \le 0.166 \text{ if } d_b = 0.5 \cdot in \\ slope_h \le 0.125 \text{ if } d_b = 0.6 \cdot in \end{vmatrix}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

holddown force at jacking, P_{hd},

$$P_{hd} := f_{pj} \cdot N_h \cdot A_p \cdot \sin\left(atan\left(\frac{R_h}{x_h + P2}\right)\right) \qquad P_{hd} = 44.8\,kip$$ (for checking shop drawing)

$$e_{Dh} := \frac{R_h}{(x_h + P2)} \cdot (x_h + P2 - D) + F_{CL} - Y_{bg} \qquad e_{Dh} = 4.18\,\text{in}$$

At top of girder,

$$\left(-\frac{M_{gD}}{S_{tg}} - \frac{P_{si}}{A_g}\right) + \frac{P_{sis} \cdot e_s - P_{sih} \cdot e_{Dh} - P_{sit} \cdot e_{temp}}{S_{tg}} = -0.5\,\text{ksi}$$

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24 \cdot \sqrt{\frac{f'_{ci}}{\text{ksi}}} \cdot \text{ksi} = 0.66\,\text{ksi}$$ **OK**

or w/o bonded reinforcement (use),

$$\min\left(\begin{pmatrix} 0.0948 \cdot \sqrt{\dfrac{f'_{ci}}{\text{ksi}}} \cdot \text{ksi} \\ 0.200 \cdot \text{ksi} \end{pmatrix}\right) = 0.2\,\text{ksi}$$ **OK**

At bottom of girder,

$$\left(\frac{M_{gD}}{S_{bg}} - \frac{P_{si}}{A_g}\right) + \frac{-P_{sis} \cdot e_s + P_{sih} \cdot e_{Dh} + P_{sit} \cdot e_{temp}}{S_{bg}} = -4.23\,\text{ksi}$$

< allowable $-0.60 \cdot f'_{ci} = -4.5\,\text{ksi}$ **OK**

11.3 Concrete Stresses at Transfer At Harping Point

Moment due to weight of girder,

$$M_{gth} := w_g \cdot \frac{GL}{2} \cdot (P2 + x_h) - \frac{w_g}{2} \cdot (P2 + x_h)^2 \qquad M_{gth} = 1.765 \times 10^3\,\text{kip} \cdot \text{ft}$$

For straight strands,

$P_{sis} = 1.017 \times 10^3\,\text{kip}$ $e_s = 34.33\,\text{in}$

For harped strands,

$P_{sih} = 625.908\,\text{kip}$ $e_h = 34.27\,\text{in}$

For temporary strands,

$P_{sit} = 78.238\,\text{kip}$ $e_{temp} = 33.48\,\text{in}$

At top of girder,

$$\left(-\frac{M_{gth}}{S_{tg}} - \frac{P_{si}}{A_g}\right) + \frac{P_{sis} \cdot e_s + P_{sih} \cdot e_h - P_{sit} \cdot e_{temp}}{S_{tg}} = -0.19\,\text{ksi}$$

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24 \cdot \sqrt{\frac{f'_{ci}}{\text{ksi}}} \cdot \text{ksi} = 0.66\,\text{ksi}$$ **OK**

or w/o bonded reinforcement (use),

$$\min\left(\begin{pmatrix} 0.0948 \cdot \sqrt{\frac{f'_{ci}}{\text{ksi}}} \cdot \text{ksi} \\ 0.200 \cdot \text{ksi} \end{pmatrix}\right) = 0.2\,\text{ksi}$$ **OK**

At bottom of girder,

$$\left(\frac{M_{gth}}{S_{bg}} - \frac{P_{si}}{A_g}\right) + \frac{-P_{sis} \cdot e_s - P_{sih} \cdot e_h + P_{sit} \cdot e_{temp}}{S_{bg}} = -4.56\,\text{ksi}$$

< allowable $-0.60 \cdot f'_{ci} = -4.5\,\text{ksi}$ **OK**

(Note: f'ci at lifting is more critical at lifting due to shifting of support points into mid-span)

12 Strength Limit State

Resistance factors (§5.5.4.2.1)

$\phi_f := 0.90$ for flexure and tension of reinforced concrete

$\phi_p := 1.00$ for flexure and tension of prestressed concrete

$\phi_v := 0.90$ for shear and torsion of normal weight concrete

Load Modifier

$\eta_D := 1.00$ for non-ductile components and connections (§1.3.3, for conventional design)

$\eta_R := 1.00$ for redundant members (§1.3.4, for conventional level of redundancy)

$\eta_I := 1.00$ for operationally important bridge (§1.3.5, for typical bridges)

$$\eta := \max\left(\begin{pmatrix} \eta_D \cdot \eta_R \cdot \eta_I \\ 0.95 \end{pmatrix}\right) \qquad \eta = 1$$ (§1.3.2)

12.1 Ultimate Moment Required

Strength I load combination - normal vehicular load without wind (§3.4.1).

The force effects due to temperature shrinkage and creep are ignored.

Load factors (LRFD Table 3.4.1-1):

$\gamma_p := 1.25$ for component and attachments

$\gamma_L := 1.75$ for LL

Flexural moment

Dead load moment,

$$M_{DC} := M_g + M_{sp}\left(\frac{L}{2}\right) + M_d + M_b \qquad M_{DC} = 4586\,kip\cdot ft$$

Live load moment,

$$M_L = 2.615 \times 10^3\,kip\cdot ft$$

$$M_u := \eta\cdot\left(\gamma_p\cdot M_{DC} + \gamma_L\cdot M_L\right) \qquad M_u = 10309\,kip\cdot ft$$

12.2 Flexural Resistance (§5.7.3)

Note: In PGSuper, moment capacity is computed using a non-linear strain-compatibility methodology.

Find stress in prestressing steel at nominal flexural resistance, f_{ps} (§5.7.3.1.1)

$$f_{pe} = 151.315\,ksi \qquad 0.5\cdot f_{pu} = 135\,ksi$$

$$\text{if}\left(f_{pe} \ge 0.5\cdot f_{pu}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$k := 2\cdot\left(1.04 - \frac{f_{py}}{f_{pu}}\right) \qquad k = 0.28$$ (LRFD Eq. 5.7.3.1.1-2)

$$A_s := 0\cdot in^2$$

$$A'_s := 0\cdot in^2$$ (conservatively)

$h_f := t_{s1}$ $\qquad h_f = 7\,in$ (depth of compression flange)

d_p, distance from extreme compression fiber to the centroid of the prestressing tendons,

$$d_p := t_{s1} + d_g - \left(Y_{bg} - e_p\right) \qquad d_p = 76.79\,in$$

$$b = 96\,in$$

$$b_w = 6\,in$$

$$\beta_1 := \text{if}\left[f'_{cs} \le 4\cdot ksi, 0.85, 0.85 - 0.05\cdot\left(\frac{f'_{cs} - 4.0\cdot ksi}{1.0\cdot ksi}\right)\right] \qquad \beta_1 := \begin{cases} \beta_1 & \text{if } \beta_1 \ge 0.65 \\ 0.65 & \text{otherwise} \end{cases}$$

$\beta_1 = 0.85$ (§5.7.2.2)

Assume flanged section,

$$c_f := \frac{A_{ps}\cdot f_{pu} - 0.85\cdot\beta_1\cdot f'_{cs}\cdot(b - b_w)\cdot h_f}{0.85\cdot f'_{cs}\cdot\beta_1\cdot b_w + k\cdot A_{ps}\cdot\frac{f_{pu}}{d_p}} \qquad c_f = 24.33\,in$$

(for flange section, ignore the girder top flange and the modular ratio of the slab and deck concrete for simplicity)

Assume rectangular section,

$$c_r := \frac{A_{ps}\cdot f_{pu}}{0.85\cdot f'_{cs}\cdot\beta_1\cdot b + k\cdot A_{ps}\cdot\frac{f_{pu}}{d_p}} \qquad c_r = 8.59\,in$$

$$c := \begin{cases} c_r & \text{if } 0\cdot in \le c_r \le h_f \\ c_f & \text{otherwise} \end{cases} \qquad c = 24.33\,in$$

Stress in prestressing steel at nominal flexural resistance, f_{ps} (§5.7.3.1.1),

$$f_{ps} := f_{pu}\cdot\left(1 - k\cdot\frac{c}{d_p}\right) \qquad f_{ps} = 246.1\,ksi$$

Flexural Resistance (§5.7.3.2.2 & 5.7.3.2.2),

$$a := \beta_1\cdot c \qquad a = 20.68\,in$$

$$M_n := \text{if}\left[h_f < c, A_{ps}\cdot f_{ps}\cdot\left(d_p - \frac{a}{2}\right) + 0.85\cdot f'_{cs}\cdot(b - b_w)\cdot\beta_1\cdot h_f\cdot\left(\frac{a}{2} - \frac{h_f}{2}\right), A_{ps}\cdot f_{ps}\cdot\left(d_p - \frac{a}{2}\right)\right]$$

$$M_n = 13455\,kip\cdot ft$$

$$M_r := \phi_p\cdot M_n \qquad M_r = 13455\,kip\cdot ft$$

$M_u \le M_r = 1$ **OK** where $M_u = 10309\,kip\cdot ft$

13 Limit for Reinforcement

13.1 Maximum Reinforcement (§5.7.3.3.1)

$$d_e := \frac{A_{ps} \cdot f_{ps} \cdot d_p}{A_{ps} \cdot f_{ps}} \qquad d_e = 76.79\,in$$

The maximum amount of prestressed and non-prestressed reinforcement shall be such that

$$\mathrm{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \text{where} \quad \frac{c}{d_e} = 0.32$$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

13.2 Minimum Reinforcement (§5.7.3.3.2)

AASHTO 9.18.2

Compressive stress in concrete due to effective prestress force (after all losses) at midspan

$$f_{peA} := \frac{P_e}{A_g} + P_e \cdot \frac{e_p}{S_{bg}} \qquad f_{peA} = 5.13\,ksi \qquad \text{(compression)}$$

Non-composite dead load moment at section, M_{dnc},

$$M_{dnc} := M_g + M_{sp}\left(\frac{L}{2}\right) + M_d \qquad M_{dnc} = 4.178 \times 10^3\,kip \cdot ft$$

$$M_{cr} := (f_r + f_{peA}) \cdot S_b - M_{dnc} \cdot \left(\frac{S_b}{S_{bg}} - 1\right) \qquad 1.2 \cdot M_{cr} = 9376\,kip \cdot ft$$

$M_r \ge 1.2 \cdot M_{cr} = 1$ **OK** where $M_r = 13455\,kip \cdot ft$

If NG, check against 1.33 Mu.

13.3 Development of Prestressing Strand (§5.11.4)

$f_{ps} = 246.05\,ksi \qquad f_{pe} = 151.32\,ksi \qquad d_b = 0.6\,in$

Pretensioning strand shall be bonded beyond the critical section for development length, L_d,

$$L_d := 1.6 \cdot \left(\frac{f_{ps}}{ksi} - \frac{2}{3} \cdot \frac{f_{pe}}{ksi}\right) \cdot d_b \qquad L_d = 11.61\,ft$$

14 Shear Design & Longitudinal Reinforcement Design

14.1 Shear Design Procedure (§5.8.1.1)

Distance from compression face to centroid of tension reinforcement,

$d := d_e$ $\quad d = 6.399\,\text{ft}$

Since $\frac{L}{2} = 65.875\,\text{ft}$ > $2 \cdot d = 12.798\,\text{ft}$ Use sectional model

14.2 Shear Force Effect (§5.8.3.2)

Compute effective shear depth (§5.8.2.7), d_v,

de and a need to be modified to reflect the critical section location, also at harping point location.

$d_e - \frac{a}{2} = 66.45\,\text{in}$

$0.9 \cdot d_e = 69.11\,\text{in}$

$0.72 \cdot (d_g + t_{s1}) = 57.96\,\text{in}$

$$d_v := \max\left[\left[\begin{array}{c} d_e - \frac{a}{2} \\ 0.9 \cdot d_e \\ 0.72 \cdot (d_g + t_{s1}) \end{array}\right]\right] \qquad d_v = 69.11\,\text{in}$$

(§5.8.3.2) The location of critical section dc for shear shall be taken as the larger of $0.5 \cdot d_v \cdot \cot(\theta)$ or d_v from the internal face of the support. Since q, the angle of diagonal compressive stress, is not known; therefore use d_v for shear force calculations.

use $d_c := d_v$ for now

Shear at d_c from CL of bearing

Dead load

girder, $V_g := w_g \cdot \left(\frac{L}{2} - d_c\right)$ $\quad V_g = 49.94\,\text{kip}$

slab+pad, $V_{sp}(x) := w_{spu} \cdot \left(\frac{L}{2} - x\right)$ $\quad V_{sp}(d_c) = 58.86\,\text{kip}$

diaphragm, $$V_d := \left| \begin{array}{ll} 1.5 \cdot P & \text{if } n_{diaph} = 3 \\ 1.0 \cdot P & \text{if } n_{diaph} = 2 \\ 0.5 \cdot P & \text{if } n_{diaph} = 1 \\ 0 \cdot \text{kip} & \text{otherwise} \end{array} \right. \qquad V_d = 5.71\,\text{kip}$$

s.i.d.l., $V_b := w_b \cdot \left(\frac{L}{2} - d_c \right)$ $V_b = 11.3\,\text{kip}$

$V_{DC} := V_g + V_{sp}(d_c) + V_d + V_b$ $V_{DC} = 125.82\,\text{kip}$

Live Load Shear

D.F. for Shear (interior girder)

Range of applicability (LRFD Table 4.6.2.2.3a-1), case k

$\text{if}(3.5\cdot\text{ft} \le S \le 16.0\cdot\text{ft}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(20\cdot\text{ft} \le L \le 240\cdot\text{ft}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}\left(4.5\cdot\text{in} \le t_{s1} \le 12.0\cdot\text{in}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$\text{if}\left(N_b \ge 4, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

For two or more design lanes loaded (interior girder):

$DF_{vi} := 0.2 + \frac{S}{12\cdot\text{ft}} - \left(\frac{S}{35\cdot\text{ft}}\right)^{2.0}$ $DF_{vi} = 0.814$

D.F. for Shear (exterior girder)

Range of applicability (LRFD Table 4.6.2.2.3b-1), case k,

$\text{if}(-1.0\cdot\text{ft} \le \text{overhang} - \text{cw} \le 5.5\cdot\text{ft}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

Correction factor,

$e4 := 0.6 + \frac{\text{overhang} - \text{cw}}{10.0\cdot\text{ft}}$ $e4 = 0.9$

D.F. for shear (exterior girder),

$DF_{ve} := e4 \cdot DF_{vi}$ $DF_{ve} = 0.733$

Correction factor for skewed bridges (Table 4.6.2.2.3c-1)

$\text{if}\left(0\cdot\text{deg} \le \theta_{sk} \le 60\cdot\text{deg}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$\text{if}(3.5\cdot\text{ft} \le S \le 16.0\cdot\text{ft}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}(20\cdot\text{ft} \le L \le 240\cdot\text{ft}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

$\text{if}\left(N_b \ge 4, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$SK_v := 1.0 + 0.20\cdot\left(\frac{L\cdot t_{s1}^{\,3}}{K_g}\right)^{0.3}\cdot\tan\left(\theta_{sk}\right)$ $SK_v = 1.108$

Increased D.F. for shear,

$$DF_v := \begin{cases} SK_v \cdot DF_{vi} & \text{if } girder = \text{"interior"} \\ (SK_v \cdot DF_{ve}) & \text{if } girder = \text{"exterior"} \end{cases} \qquad DF_v = 0.902$$

L.L. shear at critical section (see QconBridge output), $\frac{d_c}{L} = 0.044$

$$V_{LL} := 118.5 \cdot kip$$

$$V_L := V_{LL} \cdot DF_v \qquad V_L = 106.94\,kip$$

Shear force effect,

$$V_u := \eta \cdot (\gamma_p \cdot V_{DC} + \gamma_L \cdot V_L) \qquad V_u = 344.42\,kip$$

14.4.3 Determination of β and θ (§5.8.3.4.2)

V_p

a: angle of harped strands inclination

CL of bearing to end of girder, $P2 = 3.94\,in$

$$\alpha := atan\left(\frac{R_h}{x_h + P2}\right) \qquad \alpha = 3.65\,deg$$

$$P_h := f_{pe} \cdot A_p \cdot N_h \qquad P_h = 525.366\,kip$$

$$V_p := P_h \cdot sin(\alpha) \qquad V_p = 33.45\,kip$$

f_{po}, stress in prestressing steel when the stress in the surrounding concrete is 0.0,

Within the transfer length, f po shall be increased linearly from zero at the location where the bond commences to its full value at the end of transfer length.

since $d_c + P2 = 73.045\,in$ >= $D = 36\,in$

$$f_{po} := 0.7 f_{pu} \qquad f_{po} = 189\,ksi \qquad \text{(tension)}$$

N_u, external axial forces,

$$N_u := 0.0 \cdot kip$$

M_u at critical section,

$$M_g := w_g \cdot \frac{L}{2} \cdot d_c - w_g \cdot \frac{d_c^{\,2}}{2} \qquad M_g = 301.4\,kip \cdot ft$$

$M_{sp}(d_c) = 355.2\,kip \cdot ft$

$M_d := V_d \cdot d_c$ $\qquad M_d = 32.9\,kip \cdot ft$

$M_b := w_b \cdot \frac{L}{2} \cdot d_c - w_b \cdot \frac{d_c^{\,2}}{2}$ $\qquad M_b = 68.2\,kip \cdot ft$

$M_{DC} := M_g + M_{sp}(d_c) + M_d + M_b$ $\qquad M_{DC} = 757.709\,kip \cdot ft$

$M_{LL} := 759.69 \cdot kip \cdot ft$ (from QconBridge output)

("it is conservative to take Mu as the highest factored moment that will occurred at the section rather than the coincident moment.")

$M_L := M_{LL} \cdot DF$ $\qquad M_L = 475.99\,kip \cdot ft$

M_u, then,

$M_u := \eta \cdot (\gamma_p \cdot M_{DC} + \gamma_L \cdot M_L)$ $\qquad M_u = 1780\,kip \cdot ft$

$M_u := \max((M_u \;\; V_u \cdot d_v))$ $\qquad M_u = 1983\,kip \cdot ft$

A_{ps}: area of prestressing steel on the flexural tension side of the member

$A_{ps} := N_s \cdot A_p$ $\qquad A_{ps} = 5.642\,in^2$

A_s: area of non-prestressed reinforcing steel on the flexural tension side of the member (LRFD Fig. 5.8.3.4.2-3)

$A_s := 0.0 \cdot in^2$ (conservatively)

per eq. 5.8.3.4.2-3, A_c = area of concrete on the flexural tension side of the member as shown in Figure 5.8.3.4.2-1

$A_c := 6 \cdot in \cdot 19 \cdot in + 3 \cdot in \cdot 9.5 \cdot in + 6 \cdot in \cdot 0.5(d_g + t_{s1})$ $\qquad A_c = 384\,in^2$

(See design policy memo for the revised ε_{xx} equation)

$$\varepsilon_{xx} := \frac{\frac{M_u}{d_v} + 0.5 \cdot N_u + (V_u - V_p) - A_{ps} \cdot f_{po}}{2 \cdot (E_s \cdot A_s + E_p \cdot A_{ps})} \qquad \varepsilon_{xx} = -1.28 \times 10^{-3}$$

$$\varepsilon_x := \begin{cases} \varepsilon_{xx} & \text{if } \varepsilon_{xx} \ge 0.0 \\ \varepsilon_{xx} \cdot \frac{2 \cdot (E_s \cdot A_s + E_p \cdot A_{ps})}{2 \cdot (E_c \cdot A_c + E_s \cdot A_s + E_p \cdot A_{ps})} & \text{otherwise} \end{cases} \qquad \varepsilon_x = -8.14 \times 10^{-5}$$

LRFD Eq. 5.8.3.4.2-1

$b_v := b_w \qquad b_v = 6\,\text{in}$

$$v := \frac{V_u - \phi_v \cdot V_p}{\phi_v \cdot b_v \cdot d_v} \qquad v = 0.84\,\text{ksi}$$

$$\frac{v}{f'_c} = 0.099$$

From LRFD Table 5.8.3.4.2-1, with transverse reinforcement

use $\theta := 21.4 \cdot \text{deg}$

$\beta := 3.24$

Check critical section location (§5.8.3.2)

$0.5 \cdot d_v \cdot \cot(\theta) = 88.17\,\text{in}$

$d_c = 69.11\,\text{in}$ Larger governs, say OK

14.4.4 Required Shear Strength (§5.8.3.3)

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f'_c}{\text{ksi}}} \cdot \text{ksi} \cdot b_v \cdot d_v \qquad V_c = 123.77\,\text{kip}$$

$\phi_v = 0.9$

$V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p) = 1$ **if positive,** transverse reinforcement req'd (§5.8.2.4)

Try two legs #4, $A_v := 0.40 \cdot \text{in}^2$ $s := 18 \cdot \text{in}$ (see standard girder plan)

V_s: shear to be taken by shear reinforcement (§5.8.3.3)

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s} \qquad V_s = 235.1\,\text{kip} \qquad \begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix} = \begin{pmatrix} 392.34 \\ 914.565 \end{pmatrix} \text{kip}$$

$$V_n := \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix}\right) \qquad V_n = 392.34\,\text{kip}$$

$$\text{if}\left(\phi_v \cdot V_n \geq V_u, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad V_u = 344.42\,\text{kip}$$

Check minimum shear reinforcement (§5.8.2.5)

$$\text{if}\left(A_v \geq 0.0316 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot \frac{b_v \cdot s}{f_y}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad 0.0316 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot \frac{b_v \cdot s}{f_y} = 0.17\,\text{in}^2$$

Check maximum shear reinforcement spacing (§5.8.2.7)

$$s_{max} := \text{if}\left[v < 0.125 \cdot f'_c, \min\left(\begin{pmatrix} 0.8 \cdot d_v \\ 24.0 \cdot in \end{pmatrix}\right), \min\left(\begin{pmatrix} 0.4 \cdot d_v \\ 12.0 \cdot in \end{pmatrix}\right)\right] \qquad s_{max} = 24\,\text{in}$$

$$s := \min\left(\begin{pmatrix} s_{max} \\ s \end{pmatrix}\right) \qquad s = 18\,\text{in}$$

use $A_v = 0.4\,\text{in}^2$ and $s = 18\,\text{in}$

14.4.5 Longitudinal Reinforcement at Critical Section (§5.8.3.5)

$\phi_n := 0.75$ for axial compression with spirals or ties (§5.5.4.2.1)

A_{ps}: area of prestressing steel on the flexural tension side of the member

$A_{ps} \cdot f_{ps} = 1.388 \times 10^3\,\text{kip}$

(per §C5.8.3.4.2, "it is conservative to take Mu as the highest factored moment that will occurred at the section rather than the coincident moment.")

$$\frac{M_u}{d_v \cdot \phi_p} + 0.5 \cdot \frac{N_u}{\phi_n} + \left[\frac{V_u}{\phi_v} - 0.5 \cdot \min\left(\left(V_s \quad \frac{V_u}{\phi_v}\right)\right) - V_p\right] \cdot \cot(\theta) = 935.6\,\text{kip}$$

$$\text{if}\left[A_{ps} \cdot f_{ps} \geq \frac{M_u}{d_v \cdot \phi_p} + 0.5 \cdot \frac{N_u}{\phi_n} + \left[\frac{V_u}{\phi_v} - 0.5 \cdot \min\left(\left(V_s \quad \frac{V_u}{\phi_v}\right)\right) - V_p\right] \cdot \cot(\theta), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$

if true, no longitudinal reinforcement required.

14.5 Shear Design and Longitudinal Reinforcement at Harping Point

14.5.1 Factored Shear Force (§1.3.2)

Dead load girder $V_g := w_g \cdot \left(\frac{L}{2} - x_h\right)$ $V_g = 10.8\,\text{kip}$

slab + pad $V_{sp}(x_h) = 12.7\,\text{kip}$

diaphragm $V_d := \begin{cases} 0.5\cdot P & \text{if } n_{diaph} = 3 \\ 0\cdot P & \text{if } n_{diaph} = 2 \\ 0.5\cdot P & \text{if } n_{diaph} = 1 \\ 0\cdot \text{kip} & \text{otherwise} \end{cases}$ $V_d = 1.9\,\text{kip}$

s.i.d.l. $V_b := w_b\cdot\left(\frac{L}{2} - x_h\right)$ $V_b = 2.4\,\text{kip}$

$V_{DC} := V_g + V_{sp}(x_h) + V_d + V_b$ $V_{DC} = 27.8\,\text{kip}$

LL shear at harping point (see QconBridge output)

$V_{LL} := 55\cdot\text{kip}$

$V_L := V_{LL}\cdot DF_v$ $V_L = 49.6\,\text{kip}$

Shear force effect $V_u := \eta\cdot(\gamma_p\cdot V_{DC} + \gamma_L\cdot V_L)$ $V_u = 121.6\,\text{kip}$

14.5.2 Nominal Shear Resistances (§5.8.3.3)

Determination of b and q (§5.8.3.4.2)

$V_p := 0.0\cdot\text{kip}$

f_{po}, stress in prestressing steel when the stress in the surrounding concrete is 0.0,

$f_{po} := 0.70 f_{pu}$ $f_{po} = 189\,\text{ksi}$ (tension)

N_u, external axial forces,

$N_u := 0.0\cdot\text{kip}$

M_u at harping point,

$M_{DC} := M_{gh} + M_{sp}(x_h) + M_{dh} + M_{bh}$ $M_{DC} = 4393\,\text{kip}\cdot\text{ft}$

$M_{Lh} = 2522\,\text{kip}\cdot\text{ft}$

(per §C5.8.3.4.2, "it is conservative to take Mu as the highest factored moment that will occurred at the section rather than the coincident moment.")

M_u, then

$$M_u := \eta \cdot (\gamma_p \cdot M_{DC} + \gamma_L \cdot M_{Lh}) \qquad M_u = 9905\,kip \cdot ft$$

$$M_u := \max((M_u \quad V_u \cdot d_v)) \qquad M_u = 9905\,kip \cdot ft$$

A_{ps}: area of prestressing steel on the flexural tension side of the member

$$A_{ps} := N_p \cdot A_p \qquad A_{ps} = 9.114\,in^2$$

A_s: area of non-prestressed reinforcing steel on the flexural tension side of the member (LRFD Fig. 5.8.3.4.2-3)

$$A_s := 0.0 \cdot in^2$$

(See design policy memo for the revised ε_{xx} equation)

$$\varepsilon_{xx} := \frac{\frac{M_u}{d_v} + 0.5 \cdot N_u + (V_u - V_p) - A_{ps} \cdot f_{po}}{2 \cdot (E_s \cdot A_s + E_p \cdot A_{ps})} \qquad \varepsilon_{xx} = -1.28 \times 10^{-3}$$

$$\varepsilon_x := \begin{cases} \varepsilon_{xx} & \text{if } \varepsilon_{xx} \geq 0.0 \\ \varepsilon_{xx} \cdot \frac{2 \cdot (E_s \cdot A_s + E_p \cdot A_{ps})}{2 \cdot (E_c \cdot A_c + E_s \cdot A_s + E_p \cdot A_{ps})} & \text{otherwise} \end{cases} \qquad \varepsilon_x = -8.14 \times 10^{-5}$$

LRFD Eq. 5.8.3.4.2-1

$$v := \frac{V_u - \phi_v \cdot V_p}{\phi_v \cdot b_v \cdot d_v} \qquad v = 0.33\,ksi$$

$$\frac{v}{f'_c} = 0.038$$

From LRFD Table 5.8.3.4.2-1, with transverse reinforcement

use $\theta := 21 \cdot deg$

$\beta := 4.1$

14.5.3 Required Shear Strength

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot b_v \cdot d_v \qquad V_c = 156.6\,kip$$

$V_u > 0.5 \cdot \phi_v \cdot (V_c + V_p) = 1$ **if positive,** transverse reinforcement req'd (§5.8.2.4)

Try two legs #4, $A_v := 0.40 \cdot in^2$ $s := 18 \cdot in$ (see standard girder plan)

V_s: shear to be taken by shear reinforcement (§5.8.3.3)

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot \cot(\theta)}{s} \qquad V_s = 240\,kip$$

$$V_n := \min\left(\begin{pmatrix} V_c + V_s + V_p \\ 0.25 \cdot f'_c \cdot b_v \cdot d_v + V_p \end{pmatrix}\right) \qquad V_n = 396.66\,kip$$

$$\text{if}(\phi_v \cdot V_n \geq V_u, \text{"OK"}, \text{"NG"}) = \text{"OK"} \qquad V_u = 121.65\,kip$$

Check Minimum Shear Reinforcement (§5.8.2.5)

$$\text{if}\left(A_v \geq 0.0316 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot \frac{b_v \cdot s}{f_y}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad 0.0316 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \cdot \frac{b_v \cdot s}{f_y} = 0.17\,in^2$$

Check Maximum Shear Reinforcement Spacing (§5.8.2.7)

$$s_{max} := \text{if}\left[v < 0.125 \cdot f'_c, \min\left(\begin{pmatrix} 0.8 \cdot d_v \\ 24.0 \cdot in \end{pmatrix}\right), \min\left(\begin{pmatrix} 0.4 \cdot d_v \\ 12.0 \cdot in \end{pmatrix}\right)\right] \qquad s_{max} = 24\,in$$

$$s := \min\left(\begin{pmatrix} s_{max} \\ s \end{pmatrix}\right) \qquad s = 18\,in \qquad \textbf{OK}$$

use $A_v = 0.4\,in^2$ and $s = 18\,in$

14.5.4 Longitudinal Reinforcement at Harping Point (§5.8.3.5)

$$A_s \cdot f_y + A_{ps} \cdot f_{ps} = 2.243 \times 10^3\,kip$$

("it is conservative to take Mu as the highest factored moment that will occurred at the section rather than the coincident moment."; but it could be too conservative. per §C5.8.3.5, at max. moment locations, the tension in the reinforcement does not exceed that due to the maximum moment alone.)

$$\frac{M_u}{d_v \cdot \phi_p} + 0.5 \cdot \frac{N_u}{\phi_n} + \left[\frac{V_u}{\phi_v} - 0.5 \cdot \min\left(\left(V_s \quad \frac{V_u}{\phi_v}\right)\right) - V_p\right] \cdot \cot(\theta) = 1.896 \times 10^3\,kip$$

$$\text{if}\left[A_s \cdot f_y + A_{ps} \cdot f_{ps} \geq \frac{M_u}{d_v \cdot \phi_p} + 0.5 \cdot \frac{N_u}{\phi_n} + \left[\frac{V_u}{\phi_v} - 0.5 \cdot \min\left(\left(V_s \quad \frac{V_u}{\phi_v}\right)\right) - V_p\right] \cdot \cot(\theta), \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$$

if true, no longitudinal reinforcement required.

14.6 Pretension Anchorage Zone

14.6.1 Jacking Force In Service (§3.4.3)

P_j, jacking force, $P_j := f_{pj} \cdot N_p \cdot A_p$ $P_j = 1846\,kip$

check jacking force in service (§3.4.3),

V_{DC}, permanent dead reaction at bearing, $V_{DC} := w_g \cdot \frac{L}{2}$

$1.3 \cdot V_{DC} = 71.1\,kip$

$if(P_j \geq 1.3 \cdot V_{DC}, "OK", "NG") = "OK"$

14.6.2 Factored Bursting Resistance Pr (§5.10.10.1)

Add extra stirrups at beam ends:

Let $f_s := 20 \cdot ksi$ (conservatively)

A_s, total area of reinforcement located within the distance $\frac{d_g}{4} = 18.4\,in$ from the end of the beam

$$A_s := \frac{0.04 \cdot (f_{pj} \cdot A_{pstemp})}{f_s}$$ $A_s = 3.867\,in^2$

Note: the the prestressing force prior to release (see PCI design example)

use 5 #5 U- stirrups @ 3.5" spacing at each end of beams (see standard plan).

$A_s := 5 \cdot 0.31 \cdot in^2 \cdot 2$ $A_s = 3.1\,in^2$ **say OK**

14.6.2 Confinement Reinforcement (§5.10.10.2)

For the distance within $1.5 \cdot d_g = 9.19\,ft$

from the end of the beams, the reinforcement shall not be less than #3 deformed bars, with spacing not exceeding 6", and shaped to enclose the strands in the bottom flange.

Need to modify the standard plan.

15 Lift Analysis

Allowable Stresses

Lifting

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24 \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi = 0.66\,ksi$$

or w/o bonded reinforcement,

$$\min\left(\begin{pmatrix} 0.0948 \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi \\ 0.200 \cdot ksi \end{pmatrix}\right) = 0.2\,ksi$$

Allowable compression (§5.9.4.1.1)

$$-0.60 \cdot f'_{ci} = -4.5\,ksi$$

Shipping

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24 \cdot \sqrt{\frac{f'_{c}}{ksi}} \cdot ksi = 0.7\,ksi$$

or w/o bonded reinforcement,

$$\min\left(\begin{pmatrix} 0.0948 \cdot \sqrt{\frac{f'_{c}}{ksi}} \cdot ksi \\ 0.200 \cdot ksi \end{pmatrix}\right) = 0.2\,ksi$$

Allowable compression during shipping and handling (§5.9.4.2.1),

$$-0.60 \cdot f'_{c} = -5.1\,ksi$$

At lifting

Distance from end of girder to CL Brg, $P2 = 3.94\,in$

Assume pick point from the end of girder, $x_L := 2.5 \cdot ft$

Moment due to weight of girder at harping point, which is the critical section,

$$M_{gh} := w_g \cdot \frac{GL}{2} \cdot (x_h + P2 - x_L) - \frac{1}{2} \cdot w_g \cdot (x_h + P2)^2 \qquad M_{gh} = 1627\,kip \cdot ft$$

For straight strands at lifting,

$$P_{sis} = 1017\,kip$$

$$e_s = 34.33\,in$$

For harped strands at lifting,

$$P_{sih} = 625.91\,kip$$

$$e_h = 34.27\,in$$

At bottom of girder at lifting

$$f_c := \left(\frac{M_{gh}}{S_{bg}} - \frac{P_{si}}{A_g}\right) + \frac{-P_{sis}\cdot e_s - P_{sih}\cdot e_h + P_{sit}\cdot e_{temp}}{S_{bg}}$$

$f_c = -4.68\,ksi$ < allowable $-0.60\cdot f'_{ci} = -4.5\,ksi$

say OK

At top of girder at lifting

$$f_t := \left(-\frac{M_{gh}}{S_{tg}} - \frac{P_{si}}{A_g}\right) + \frac{P_{sis}\cdot e_s + P_{sih}\cdot e_h - P_{sit}\cdot e_{temp}}{S_{tg}} \qquad f_t = -0.08\,ksi$$

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24\cdot\sqrt{\frac{f'_{ci}}{ksi}}\cdot ksi = 0.66\,ksi$$ **OK**

or w/o bonded reinforcement

$$\min\left(\begin{pmatrix} 0.0948\cdot\sqrt{\frac{f'_{ci}}{ksi}}\cdot ksi \\ 0.200\cdot ksi \end{pmatrix}\right) = 0.2\,ksi$$ **OK**

AT Shipping

Per standard specs 6-02.3(25)M, girder support during shipping shall meet these requirements unless otherwise shown in the plans:

Series W42G and W50MG and all bulb tee girders	$3\cdot ft$
Series W58G	$4\cdot ft$
Series W74G	$5\cdot ft$
Series W83G and W95G	$8\cdot ft$

Assume shipping support from the either end of girder, $x_S := 5\cdot ft$

Moment due to weight of girder at harping point, which is the critical section,

$$M_{gh} := w_g \cdot \frac{GL}{2} \cdot \left(x_h + P2 - x_S\right) - \frac{1}{2} \cdot w_g \cdot \left(x_h + P2\right)^2 \qquad M_{gh} = 1.489 \times 10^3 \, kip \cdot ft$$

Prestress loss (per BDM 6.2.3.C and design memo), $LOSS_s := 0.75 \cdot \Delta f_{pT}$ $\quad LOSS_s = 36.904\,ksi$

For straight strands at shipping,

$$P_{ss} := N_s \cdot A_p \cdot \left(f_{pj} - LOSS_s\right) \qquad P_{ss} = 934.294\,kip$$

For harped strands at shipping,

$$P_{sh} := N_h \cdot A_p \cdot \left(f_{pj} - LOSS_s\right) \qquad P_{sh} = 575\,kip$$

For temporary strands at shipping,

$$P_{st} := N_t \cdot A_p \cdot \left(f_{pj} - LOSS_s\right) \qquad P_{st} = 71.869\,kip$$

$$P_s := P_{ss} + P_{sh} + P_{st} \qquad P_s = 1581.1\,kip$$

Two possible shipping conditions:

I. 20% Impact (dead load, up or down) (also see §5.14.1.2.1 for 50% requirement)
II. 6% superelevation of road (normally use 6%)

set $se := 0.06$

Bottom Flange

I. 20% Impact

At bottom of girder at shipping (impact up 20%)

$$f_c := \left(\frac{0.8 \cdot M_{gh}}{S_{bg}} - \frac{P_s}{A_g}\right) + \frac{-P_{ss} \cdot e_s - P_{sh} \cdot e_h + P_{st} \cdot e_{temp}}{S_{bg}} \qquad f_c = -4.55\,ksi$$

At bottom of girder at shipping (impact down 20%)

$$\left(\frac{1.2 \cdot M_{gh}}{S_{bg}} - \frac{P_s}{A_g}\right) + \frac{-P_{ss} \cdot e_s - P_{sh} \cdot e_h + P_{st} \cdot e_{temp}}{S_{bg}} = -4.05\,ksi$$

II. Superelevation of road

$I_y := 35394 \cdot in^4$ approx. about weak axis (from PGSuper output)

Bottom flange width, $b_{bf} := 25 \cdot in$

At uphill bottom flange of girder at shipping,

$$f_c := \left(\frac{M_{gh}}{S_{bg}} - \frac{P_s}{A_g}\right) + \frac{-P_{ss} \cdot e_s - P_{sh} \cdot e_h + P_{st} \cdot e_{temp}}{S_{bg}} - \frac{se \cdot M_{gh} \cdot \left(0.5 \cdot b_{bf}\right)}{I_y} \qquad f_c = -4.68\,ksi \quad \text{(governs)}$$

< allowable $-0.60 \cdot f'_c = -5.1\,ksi$ **OK**

Top Flange

I. 20% Impact

At top of girder at shipping (impact up 20%),

$$\left(-\frac{0.8\cdot M_{gh}}{S_{tg}} - \frac{P_s}{A_g}\right) + \frac{P_{ss}\cdot e_s + P_{sh}\cdot e_h - P_{st}\cdot e_{temp}}{S_{tg}} = 0.16\,\text{ksi}$$

At top of girder at shipping (impact down 20%),

$$\left(-\frac{1.2\cdot M_{gh}}{S_{tg}} - \frac{P_s}{A_g}\right) + \frac{P_{ss}\cdot e_s + P_{sh}\cdot e_h - P_{st}\cdot e_{temp}}{S_{tg}} = -0.3\,\text{ksi}$$

II. Superelevation of road

top flange width, $b_f = 3.583\,\text{ft}$

At downhill top flange of girder at shipping,

$$f_t := \left(-\frac{M_{gh}}{S_{tg}} - \frac{P_s}{A_g}\right) + \frac{P_{ss}\cdot e_s + P_{sh}\cdot e_h - P_{st}\cdot e_{temp}}{S_{tg}} + \frac{se\cdot M_{gh}\cdot(0.5\cdot b_f)}{I_y} \qquad f_t = 0.58\,\text{ksi} \qquad \text{(governs)}$$

Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2),

$$0.24\cdot\sqrt{\frac{f'_c}{\text{ksi}}}\cdot\text{ksi} = 0.7\,\text{ksi}$$

OK; If NG, use temporary pre- or post-tensioning.

or w/o bonded reinforcement,

$$\min\left(\begin{pmatrix} 0.0948\cdot\sqrt{\dfrac{f'_c}{\text{ksi}}}\cdot\text{ksi} \\ 0.200\cdot\text{ksi} \end{pmatrix}\right) = 0.2\,\text{ksi}$$

NG

tension area from top of girder,

$$x := \frac{f_t}{f_t - f_c}\cdot d_g \qquad x = 8.08\,\text{in}$$

> 5.5 in. (thickness of top flange)

total tensile force (conservative), $T := f_t\cdot\frac{x}{2}\cdot b_f$ $\qquad T = 100.5\,\text{kip}$

rebar required, $\frac{1.2T}{f_y} = 2.01\,\text{in}^2$ standard plan uses 4-#6 & 2-#4, $A_s := 2.16\cdot\text{in}^2$

say OK

16 Girder Stability

Deflection β_y,

$$\beta_y := \frac{w_g \cdot GL \cdot (GL - 2 \cdot x_L)^3}{384 \cdot E_{ci} \cdot I_y} \cdot \left[\frac{5}{GL} \cdot (GL - 2 \cdot x_L) - \frac{24}{GL} \cdot \left(\frac{x_L^2}{GL - 2 \cdot x_L} \right) \right] \qquad \beta_y = 24.5\,in$$

$$FS := \frac{Y_{tg}}{0.64 \cdot \beta_y} \qquad FS = 2.26$$

> 2 say OK (see PCI Journal, Vol. 16, No. 3, May-June 1971, pp. 7-9)

If NG, attach temporary lateral bracing to compression flange or provide strongbacks, stiffening trusses, pipe frames or rigidly attached lifting yokes.

The tilt ratio, r_{tilt}, (see January 1988, Univ. of Texas at Austin report CTR 3-2-84-381-4F)

where bottom flange width, $b_{bf} = 25\,in$

$$r_{tilt} := \frac{Y_{bg}}{0.5 \cdot b_{bf}} \qquad r_{tilt} = 3.04$$

(tilt ratio = 3 is relatively large)

The tilt ratio is relatively large, girders shall be braced laterally to prevent tipping or buckling.

17 Deflection and Camber (§5.7.3.6.2)

17.1 Camber Induced by Prestress at Transfer (moment area method)

Moment induced by harped strands

At end,

c.g. of girder to CL of harped strands $e_1 := Y_{tg} - F_o$ $\qquad e_1 = 6.5\,in$

$$M_a := -p_{st} \cdot A_p \cdot N_h \cdot e_1 \qquad M_a = -338\,kip \cdot ft$$

At harping point,

c.g. of girder to CL of harped strands, $e_h = 34.27\,in$

$$M_b := p_{st} \cdot A_p \cdot (N_h \cdot e_h) \qquad M_b = 1787\,kip \cdot ft$$

Moment induced by straight/temporary strands

At end and harping point,

c.g. of girder to CL of straight strands, $e_s = 34.33\,in$

c.g. of girder to CL of temporary strands, $e_{temp} = 33.48\,in$

$$M_c := p_{st} \cdot A_p \cdot (N_s \cdot e_s - N_t \cdot e_{temp}) \qquad M_c = 2691 \text{ kip} \cdot \text{ft}$$

$$M_1 := M_c + M_b \qquad M_1 = 4479 \text{ kip} \cdot \text{ft}$$

$$M_2 := M_c + M_a \qquad M_2 = 2353 \text{ kip} \cdot \text{ft}$$

then,

$$C_{ps} := \frac{GL^2}{8 \cdot E_{ci} \cdot I_g} \cdot \left[M_1 + \frac{M_2 - M_1}{3} \cdot \left(\frac{2 \cdot x_h}{L} \right)^2 \right] \qquad C_{ps} = 4.86 \text{ in} \qquad \text{(upward)}$$

Moment induced by due to release of temporary strands at bridge site

At end and harping point,

$$M_c := p_{st} \cdot A_p \cdot (N_t \cdot e_{temp}) \qquad M_c = 218.285 \text{ kip} \cdot \text{ft}$$

then,

$$C_{tps} := \frac{GL^2}{8 \cdot E_c \cdot I_g} \cdot (M_c) \qquad C_{tps} = 0.25 \text{ in} \qquad \text{(upward)}$$

17.2 Deflection due to Dead Load

girder $$\Delta_g := \frac{5 \cdot w_g \cdot L^4}{384 \cdot E_{ci} \cdot I_g} \qquad \Delta_g = 1.78 \text{ in} \qquad \text{(downward)}$$

slab+pad $$\Delta_{sp} := \left(\frac{5 \cdot w_{spu}}{384} \right) \cdot \frac{L^4}{E_c \cdot I_g} \qquad \Delta_{sp} = 1.97 \text{ in} \qquad \text{(downward)}$$

diaphragm (AISC P.4-189, coeff. e) $e_{coeff} = 0.05$

$$\Delta_d := \frac{e_{coeff} \cdot P \cdot (L)^3}{E_c \cdot I_g} \qquad \Delta_d = 0.22 \text{ in} \qquad \text{(downward)}$$

s.i.d.l. $$\Delta_b := \frac{5 \cdot w_b \cdot L^4}{384 \cdot E_c \cdot I_c} \qquad \Delta_b = 0.21 \text{ in} \qquad \text{(downward)}$$

screed setting dimension "C" $C := \Delta_{sp} + \Delta_b$ $C = 2.18 \text{ in}$

17.3 Net Deflection at Release

$C_{ps} - \Delta_g = 3.08 \text{ in}$ + upward

17.4 Deflection at Erection

"D" dimension (per BDM §6.1.8)

$$D := \begin{cases} 1.70 \cdot C_{ps} - 1.75 \cdot \Delta_g & \text{if } f'_c > 7 \cdot ksi \\ 1.80 \cdot C_{ps} - 1.85 \cdot \Delta_g & \text{otherwise} \end{cases} \qquad D = 5.15\,in$$

(+ upward)

Excess girder camber all. for establishing "A" dimension $D - C - \Delta_d + C_{tps} = 2.99\,in$

17.5 Deflection due to Live Load

The live load deflection shall be limited to (§2.5.2.6.2)

$$\frac{1}{800} \cdot L = 1.98\,in$$

The vehicular load shall included the dynamic allowance.

The live load deflection should be taken as the larger of (§3.6.1.3.2)

That resulting from the design truck alone, or
that resulting from 25% of the design truck taken together with the design lane load

The provision of §3.6.1.1.2 (multiple presence of live load) shall applied.

For a multi-girder bridge, the deflection shall be taken as deflection per lane times the number of lanes divided by the number of girders (§C2.5.2.6.2).

Estimate max. live load deflection at midspan with heavy truck axles closely spaced and centered in span.

For 2 heavy truck axles, $P := 32 \cdot kip$

$$a := \frac{L}{2} - 7 \cdot ft \qquad a = 58.875\,ft$$

$$\Delta_1 := \frac{P \cdot a}{24 \cdot E_c \cdot I_c} \cdot \left(3 \cdot L^2 - 4 \cdot a^2\right) \qquad \Delta_1 = 0.86\,in$$

(AISC 2nd ed. p. 4-192)

For 8 kip axle $P := 8 \cdot kip$

use moment area method $a := \frac{L}{2} - 21 \cdot ft$ $\quad a = 44.875\,ft$ $\quad b := L - a$

$$M := \frac{P \cdot a \cdot (L - a)}{L} \qquad M = 2841\,kip \cdot in$$

max. moment

$$M_m := \frac{M}{L - a} \cdot \frac{L}{2} \qquad M_m = 2154\,kip \cdot in$$

moment at midspan

$$R_A := \frac{\frac{1}{2} \cdot M \cdot a \cdot \left(2 \cdot \frac{a}{3}\right) + \frac{1}{2} \cdot M \cdot b \cdot \left(a + \frac{b}{3}\right)}{L} \qquad R_A = 1.003 \times 10^6\,kip \cdot in^2$$

Deflection at midspan

$$\Delta_2 := \frac{R_A \cdot \frac{L}{2} - \frac{1}{2} \cdot M_m \cdot \frac{L}{2} \cdot \frac{L}{4}}{E_c \cdot I_c} \qquad \Delta_2 = 0.08\,in$$

Then, $\Delta_{hs20} := \Delta_1 + \Delta_2$ $\qquad \Delta_{hs20} = 0.93\,in$ $\qquad$ per lane

Deflection due to design truck alone (per girder + impact), (ignore multiple presence factor) $\qquad \Delta_{hs20} := \Delta_{hs20} \cdot \frac{N_L}{N_b} \cdot (1 + IM)$

$$if\left(\frac{1}{800} \cdot L \geq \Delta_{hs20}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$ where $\Delta_{hs20} = 0.74\,in$

Deflection due to lane load

$$\Delta_{lane} := \frac{5 \cdot w_{lane} \cdot L^4}{384 \cdot E_c \cdot I_c} \qquad \Delta_{lane} = 0.72\,in$$ per lane

$$\Delta_{lane} := \Delta_{lane} \cdot \frac{N_L}{N_b} \qquad \Delta_{lane} = 0.43\,in$$ per girder

Deflection due to 25% truck + lane $\qquad 0.25 \cdot \Delta_{hs20} + \Delta_{lane} = 0.62\,in$

$$if\left(\frac{1}{800} \cdot L \geq 0.25 \cdot \Delta_{hs20} + \Delta_{lane}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

18 Detail of Reinforcement

18.1 Min. spacing of prestressing tendons (§5.10.3.3.1)

The clear distance between pretensioning strands, including shielded ones, at the end of a member with the development length, for each strand shall not be less than:

$$\max\left[\begin{bmatrix} 3.0 \cdot d_b \\ 1.33 \cdot (0.375 \cdot in) \end{bmatrix}\right] = 1.8\,in$$

where the nominal diameter of the strands, $\qquad d_b = 0.6\,in$

and max. size of the aggregate is 0.375 in

The clear distance between strands at the end of a member may be decreased, if justified by performance tests of full-scale prototypes of the design.

Clear spacing used in design,

$2 \cdot in - d_b = 1.4\,in$ $\qquad$ **say OK**

Design Example 2 Cast-in-Place Concrete Slab Design

1 Structure

Design span $L := 113.911 \cdot ft$

Roadway width $BW := 25 \cdot ft$ barrier face to barrier face

Girder spacing $S := 6.75 \cdot ft$

Skew angle $\theta := 14.96 \cdot deg$

no. of girder $N_b := 4$

curb width on deck, $cw := 11.5 \cdot in$

Deck overhang (centerline of exterior girder to end of deck) $overhang := \frac{BW - (N_b - 1) \cdot S}{2} + cw$ $overhang = 3.333\,ft$

2 Criteria and assumptions

2.1 Design Live Load for Decks

(§3.6.1.3.3, not for empirical design method) Where deck is designed using the approximate strip method, specified in §4.6.2.1, the live load shall be taken as the wheel load of the 32.0 kip axle of the design truck, without lane load, where the strips are transverse.

$if(S \le 15 \cdot ft, "OK", "NG") = "OK"$ (§3.6.1.3.3)

The design truck or tandem shall be positioned transversely such that the center of any wheel load is not closer than (§3.6.1.3.1)

for the design of the deck overhang - 1 ft from the face of the curb or railing , and

for the design of all other components - 2 ft from the edge of the design lane.

(§3.6.1.3.4) For deck overhang design with a cantilever, not exceeding 6.0 ft from the centerline of the exterior girder to the face of a continuous concrete railing, the wheel loads may be replaced with a uniformly distributed line load of 1.0 KLF intensity, located 1 ft from the face of the railing.

$if(overhang - cw \le 6 \cdot ft, "OK", "NG") = "OK"$

Horizontal loads on the overhang resulting from vehicle collision with barriers shall be considered in accordance with the yield line analysis.

2.2 Dynamic Load Allowance (impact)

$IM := 0.33$ (§3.6.2.1)

2.3 Minimum Depth and Cover (§9.7.1)

slab design thickness $t_{s1} := 7 \cdot in$

for D.L. calculation $t_{s2} := 7.5 \cdot in$

min. depth $if(t_{s1} \ge 7.0 \cdot in, "OK", "NG") = "OK"$

top cover for epoxy-coated main reinforcing steel = 1.5 in. (up to #11 bar) (§5.12.4 & Table 5.12.3-1)
bottom concrete cover (unprotected main reinforcing) = 1 in. (up to #11 bar)
sacrificial thickness = 0.5 in. (§2.5.2.4)

2.4 Skew Deck (§9.7.1.3)

$\theta \leq 25 \cdot deg = 1$ **it true,** the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components.

3 Material Properties

3.1 Concrete

$f'_c := 4 \cdot ksi$ Use **CLASS 4000D** for bridge concrete deck (BDM 5.1.1)

$f_r := 0.24 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi$ $f_r = 0.48\,ksi$ (§5.4.2.6)

$w_c := 0.160 \cdot kcf$

$E_c := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi$ $E_c = 4.224\,10^3 \cdot ksi$ (§5.4.2.4)

3.2 Reinforcing Steel (§5.4.3)

$f_y := 60 \cdot ksi$ $E_s := 29000 \cdot ksi$

4 Methods of Analysis

Concrete deck slabs may be analyzed by using

Approximate elastic methods of analysis, or
Refined methods of analysis, or
Empirical design.

Per office practice, concrete deck slab shall be designed and detailed for both empirical and traditional design methods.

5 Empirical Design (§9.7.2)

5.1 Limit States (§9.5.1)

For other than the deck overhang, where empirical design is used, a concrete deck maybe assumed to satisfy service, fatigue and fracture and strength limit states requirements.

Empirical design shall not be applied to overhangs (§9.7.2.2).

5.2 Design Conditions (§9.7.2.4)

For the purpose of empirical design method, the effective length S_{eff} shall be taken as (§9.7.2.3),

web thickness $b_w := 6 \cdot in$

top flange width $b_f := 25 \cdot in$ $\quad b_f = 25\,in$

$$S_{eff} := S - b_f + \frac{b_f - b_w}{2} \qquad S_{eff} = 5.46\,ft$$

$$\text{if}\left(18.0 \ge \frac{S_{eff}}{t_{s1}} \ge 6.0, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{S_{eff}}{t_{s1}} = 9.357$$

core depth $\text{if}\left(t_{s2} - 2.5 \cdot in - 1 \cdot in \ge 4 \cdot in, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$$\text{if}\left(S_{eff} \le 13.5 \cdot ft, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

$$\text{if}\left(overhang \ge 3 \cdot t_{s1}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad overhang = 40\,in \qquad 3 \cdot t_{s1} = 21\,in$$

$$\text{if}\left(f'_c \ge 4 \cdot ksi, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

a structurally continuous concrete barrier is made composite with the overhang,

Composite construction **for steel girder (N/A)**

A minimum of two shear connectors at 2 ft centers shall be provided in the M- region of continuous steel superstructures.

5.3 Optional deflection criteria for span-to-depth ratio (LRFD Table 2.5.2.6.3.1)

Min. Depth (continuous span) where $S_{eff} = 5.458\,ft$ (slab span length):

$$\text{if}\left[\max\left(\begin{pmatrix} \dfrac{S_{eff} + 10 \cdot ft}{30} \\ 0.54 \cdot ft \end{pmatrix}\right) \le t_{s2}, \text{"OK"}, \text{"NG"}\right] = \text{"OK"} \qquad \max\left(\begin{pmatrix} \dfrac{S_{eff} + 10 \cdot ft}{30} \\ 0.54 \cdot ft \end{pmatrix}\right) = 6.5\,in$$

5.4 Reinforcement Requirement (§9.7.2.5)

Four layers of reinforcement is required in empirically designed slabs.

The amount of deck reinforcement shall be (§C9.7.2.5)

$0.27\ in^2/ft$ for each bottom layer (0.3% of the gross area of 7.5 in. slab)

$0.18\ in^2/ft$ for each top layer (0.2% of the gross area)

Try #5 @ 14 in. for bottom longitudinal and transverse, $0.31 \cdot in^2 \cdot \dfrac{1 \cdot ft}{14 \cdot in} = 0.27\,in^2$ per ft

#4 @ 12 in. for top longitudinal and transverse. $0.2 \cdot in^2 \cdot \dfrac{1 \cdot ft}{12 \cdot in} = 0.2\,in^2$ per ft

Spacing of steel shall not exceed 18 in.

$\text{if}(\theta \geq 25 \cdot \text{deg}, \text{"OK"}, \text{"NG"}) = \text{"NG"}$

if OK, double the specified reinforcement in the end zones, taken as a longitudinal distance equal to S_{eff}.

6 Traditional Design

6.1 Design Assumptions for Approx. Method of Analysis (§4.6.2)

Deck shall be subdivided into strips perpendicular to the supporting components (§4.6.2.1.1).
Continuous beam with span length as center to center of supporting elements (§4.6.2.1.6).
Wheel load may be modeled as concentrated load or load based on tire contact area.
Strips should be analyzed by classical beam theory.

6.2 Width of Equivalent Interior Strip (§4.6.2.1.3)

Strip width calculations are not needed since live load moments from Table A4-1 are used.

Spacing in secondary direction (spacing between diaphragms):

$L_d := \frac{L}{3}$ $\qquad$ $L_d = 37.97\,\text{ft}$

Spacing in primary direction (spacing between girders):

$S = 6.75\,\text{ft}$

Since $\text{if}\left(\frac{L_d}{S} \geq 1.50, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$, where $\frac{L_d}{S} = 5.63$ (§4.6.2.1.5)

Therefore, all the wheel load shall be applied to primary strip. Otherwise, the wheels shall be distributed between intersecting strips based on the stiffness ratio of the strip to sum of the strip stiffnesses of intersecting strips.

6.3 Limit States (§5.5.1)

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied.
Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck (§13.6.2).
Fatigue need not be investigated for concrete deck slabs in multi-girder applications (§5.5.3.1).

6.4 Strength Limit States

Resistance factors (§5.5.4.2.1)

$\phi_f := 0.90$ for flexure and tension of reinforced concrete

$\phi_v := 0.90$ for shear and torsion

Load Modifier

$\eta_D := 1.00$ for conventional design (§1.3.3)

$\eta_R := 1.00$ for conventional level of redundancy (§1.3.4)

$\eta_I := 1.00$ for typical bridges (§1.3.5)

$\eta := \max\left(\begin{pmatrix} \eta_D \cdot \eta_R \cdot \eta_I \\ 0.95 \end{pmatrix}\right)$ $\eta = 1$ (§1.3.2)

Strength I load combination - normal vehicular load without wind (§3.4.1)

Load factors (LRFD Table 3.4.1-1&2):

$\gamma_p := 1.25$ for component and attachments

$\gamma_L := 1.75$ for LL

Multiple presence factor (§3.6.1.1.2):

$M_1 := 1.20$ 1 truck

$M_2 := 1.00$ 2 trucks

$M_3 := 0.85$ 3 trucks (Note; 3 trucks never control for girder spacings up to 15.5 ft, per training notes)

6.4.1 Moment Force Effects Per Strip (§4.6.2.1.6)

The design section for negative moments and shear forces may be taken as follows:

Prestressed girder - shall be at 1/3 of flange width < 15 in.

Steel girder - 1/4 of flange width from the centerline of support.

Concrete box beams - at the face of the web.

web thickness $b_w = 6\,in$

top flange width $b_f = 25\,in$

Design critical section for negative moment and shear shall be at d_c, (§4.6.2.1.6)

$d_c := \min\left(\begin{pmatrix} \frac{b_f}{3} & 15 \cdot in \end{pmatrix}\right)$ $d_c = 8.33\,in$ from CL of girder (may be too conservative, see training notes)

Maximum factored moments **per unit width** based on Table A4-1: for $S = 6.75\,ft$

(include multiple presence factors and the dynamic load allowance)

applicability if [min((0.625·S 6·ft)) ≥ overhang − cw ,"OK" ,"NG"] = "OK"

if $(N_b \geq 3$,"OK" ,"NG=") = "OK"

$M_{LLp} := 5.10 \cdot \frac{kip \cdot ft}{ft}$

$M_{LLn} := 4.00 \cdot \frac{kip \cdot ft}{ft}$ (max. -M at d_c from CL of girder)

Dead load moment (STRUDL s-dl output)

$M_{DCp} := 0.56 \cdot \frac{kip \cdot ft}{ft}$ (max. +M, conservative)

$M_{DCn} := 0.008 \cdot \frac{kip \cdot ft}{ft}$ (max. -M at d_c at interior girder, conservative) $\frac{d_c}{S} = 0.103$

Factored positive moment per ft

$M_{up} := \eta \cdot (\gamma_p \cdot M_{DCp} + \gamma_L \cdot M_{LLp})$ $M_{up} = 9.62 \frac{kip \cdot ft}{ft}$

Factored negative moment

$M_{un} := \eta \cdot (\gamma_p \cdot M_{DCn} + \gamma_L \cdot M_{LLn})$ $M_{un} = 7.01 \frac{kip \cdot ft}{ft}$

6.4.2 Flexural Resistance

Normal flexural resistance of a rectangular section may be determined by using equations for a flanged section in which case b_w shall be taken as b (§5.7.3.2.3).

$$\beta_1 := \text{if}\left[f'_c \le 4 \cdot ksi, 0.85, 0.85 - 0.05 \cdot \left(\frac{f'_c - 4.0 \cdot ksi}{1.0 \cdot ksi}\right)\right]$$

$$\beta_1 := \begin{vmatrix} \beta_1 & \text{if } \beta_1 \ge 0.65 \\ 0.65 & \text{otherwise} \end{vmatrix}$$

$\beta_1 = 0.85$ (§5.7.2.2)

6.4.3 Design for Positive Moment Region

assume bar # $bar_p := 5$

$$dia(bar) := \begin{vmatrix} 0.5 \cdot in & \text{if } bar = 4 \\ 0.625 \cdot in & \text{if } bar = 5 \\ 0.75 \cdot in & \text{if } bar = 6 \end{vmatrix}$$

$d_p := t_{s1} - 1 \cdot in - \frac{dia(bar_p)}{2}$ $d_p = 5.7\,in$

$A_s := \frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_p - \sqrt{d_p^{\,2} - \frac{2 \cdot M_{up} \cdot ft}{0.85 \cdot \phi_f \cdot f'_c \cdot ft}}\right)$ $A_s = 0.4\,in^2$ per ft

use (bottom-transverse) # $bar_p = 5$ $s_p := 9 \cdot in$ (max. spa. 12 in. per BDM memo)

$$A_b(bar) := \begin{vmatrix} 0.20 \cdot in^2 & \text{if } bar = 4 \\ 0.31 \cdot in^2 & \text{if } bar = 5 \\ 0.44 \cdot in^2 & \text{if } bar = 6 \end{vmatrix}$$

$$A_{sp} := A_b(bar_p)\cdot\frac{1\cdot ft}{s_p} \qquad A_{sp} = 0.41\,in^2 \qquad \text{per ft}$$

Check max. reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where $d_e := d_p$

$$c := \frac{A_{sp}\cdot f_y}{0.85\cdot\beta_1\cdot f'_c\cdot 1\cdot ft} \qquad c = 0.72\,in$$

$$\text{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{c}{d_e} = 0.126$$

The section is not over-reinforced. Over-reinforced concrete sections shall not be permitted.

Check min. reinforcement (§5.7.3.3.2),

$$M_{cr} := f_r\cdot\frac{1}{6}\cdot 12\cdot in\cdot t_{s2}^{\,2} \qquad 1.2\cdot M_{cr} = 5.4\,kip\cdot ft \qquad M_{up}\cdot ft = 9.625\,kip\cdot ft$$

$$\text{if}\left(M_{up}\cdot ft \ge 1.2\cdot M_{cr}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

6.4.4 Design for Negative Moment Region

assume bar # $bar_n := 5$

$$d_n := t_{s1} - 2.0\cdot in - \frac{dia(bar_n)}{2} \qquad d_n = 4.69\,in$$

$$A_s := \frac{0.85\cdot f'_c\cdot ft}{f_y}\cdot\left(d_n - \sqrt{d_n^{\,2} - \frac{2\cdot M_{un}\cdot ft}{0.85\cdot\phi_f\cdot f'_c\cdot ft}}\right) \qquad A_s = 0.35\,in^2 \qquad \text{per ft}$$

use (top-transverse) bar # $bar_n = 5$ $\quad s_n := 9\cdot in$ (max. spa. 12 in. per BDM memo)

$$A_{sn} := A_b(bar_n)\cdot\frac{1\cdot ft}{s_n} \qquad A_{sn} = 0.41\,in^2 \qquad \text{per ft}$$

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where $d_e := d_n$

$$c := \frac{A_{sn}\cdot f_y}{0.85\cdot\beta_1\cdot f'_c\cdot 1\cdot ft} \qquad c = 0.72\,in$$

$$\text{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{c}{d_e} = 0.153$$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

$$M_{cr} := f_r \cdot \frac{1}{6} \cdot 12 \cdot in \cdot t_{s2}^{\,2} \qquad 1.2 \cdot M_{cr} = 5.4\,kip \cdot ft \qquad M_{un} \cdot ft = 7.01\,kip \cdot ft$$

$$\text{if}\left(M_{un} \cdot ft \ge 1.2 \cdot M_{cr}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$$

6.5 Control of Cracking by Distribution of Reinforcement (§5.7.3.4)

Service I load combination is to be considered for crack width control (§3.4.1).

Combined limit state load modifier (§1.3.2)

$$\eta_s := 1$$

Load factors (LRFD Table 3.4.1-1):

$\gamma_p := 1.00$ for component and attachments

$\gamma_L := 1.00$ for LL

$$M_{sp} := \eta_s \cdot \left(\gamma_p \cdot M_{DCp} + \gamma_L \cdot M_{LLp}\right) \qquad M_{sp} = 5.66\,\frac{kip \cdot ft}{ft}$$

$$M_{sn} := \eta_s \cdot \left(\gamma_p \cdot M_{DCn} + \gamma_L \cdot M_{LLn}\right) \qquad M_{sn} = 4.01\,\frac{kip \cdot ft}{ft}$$

$$\rho_p := \frac{A_{sp}}{d_p \cdot 12 \cdot in} \qquad \rho_n := \frac{A_{sn}}{d_p \cdot 12 \cdot in}$$

$$n := \frac{E_s}{E_c} \qquad n = 6.866 \qquad n := \max[\,[\,\text{ceil}((n - 0.495)) \;\; 6\,]\,]$$

set $n = 7$ (round to nearest integer, §5.7.1, not less than 6)

$$k(\rho) := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n \qquad k(\rho_p) = 0.252$$

$$j(\rho) := 1 - \frac{k(\rho)}{3} \qquad j(\rho_p) = 0.916$$

$$f_{sa} := \frac{M_{sp} \cdot ft}{A_{sp} \cdot j(\rho_p) \cdot d_p} \qquad f_{sa} = 31.54\,ksi$$

$Z_p := 170 \cdot \frac{kip}{in}$ crack width parameter in moderate exposure condition

$Z_n := 130 \cdot \frac{kip}{in}$ crack width parameter in severe exposure condition

for $bar_p = 5$ $s_p = 9\,in$

$d_c := (1 \cdot in) + \frac{dia(bar_p)}{2}$ $d_c = 1.3\,in$ (clear cover used to compute d_c should not be taken grater than 2 in., **OK**)

$A := 2 \cdot (d_c) \cdot s_p$ $A = 23.63\,in^2$

$$\text{if}\left[\min\left[\left[\frac{\frac{Z_p}{(d_c \cdot A)^{\frac{1}{3}}}}{0.6 \cdot f_y}\right]\right] \geq f_{sa}, \text{"OK"}, \text{"NG"}\right] = \text{"OK"} \quad \text{where} \quad \min\left[\left[\frac{\frac{Z_p}{(d_c \cdot A)^{\frac{1}{3}}}}{0.6 \cdot f_y}\right]\right] = 36\,ksi$$

$k(\rho_n) = 0.252$ $j(\rho_n) = 0.916$

$f_{sa} := \frac{M_{sn} \cdot ft}{A_{sn} \cdot j(\rho_n) \cdot d_n}$ $f_{sa} = 27.1\,ksi$

for $bar_n = 5$ $s_n = 9\,in$

$d_c := 2 \cdot in + \frac{dia(bar_n)}{2}$ $d_c = 2.31\,in$ (clear cover used to compute d_c should not be taken grater than 2 in., **OK**)

$A := 2 \cdot (d_c) \cdot s_n$ $A = 41.63\,in^2$

$$\text{if}\left[\min\left[\left[\frac{\frac{Z_n}{(d_c \cdot A)^{\frac{1}{3}}}}{0.6 \cdot f_y}\right]\right] \geq f_{sa}, \text{"OK"}, \text{"NG"}\right] = \text{"OK"} \quad \text{where} \quad \min\left[\left[\frac{\frac{Z_n}{(d_c \cdot A)^{\frac{1}{3}}}}{0.6 \cdot f_y}\right]\right] = 28.4\,ksi$$

Say OK

6.6 Shrinkage and Temperature Reinforcement (§5.10.8.2)

For components less than 48 in. thick,

where $A_g := t_{s2} \cdot 1 \cdot ft$

$A_{tem} := 0.11 \cdot \frac{A_g \cdot ksi}{f_y}$ $A_{tem} = 0.17\,in^2$ per ft

The spacing of this reinforcement shall not exceed $3 \cdot t_{s1} = 21\,in$ or 18 in (**per BDM memo 12 in.)**

top longitudinal - $bar := 4$ $s := 12 \cdot in$ $A_s := A_b(bar) \cdot \frac{1 \cdot ft}{s}$ $A_s = 0.2\,in^2$ per ft **OK**

6.7 Distribution of Reinforcement (§9.7.3.2)

The effective span length S_{eff} shall be taken as (§9.7.2.3):

$S_{eff} = 5.458\,ft$

For primary reinforcement perpendicular to traffic:

$$percent := \min\left(\left(\frac{220}{\sqrt{\frac{S_{eff}}{ft}}} \quad 67\right)\right) \qquad percent = 67$$

Bottom longitudinal reinforcement (**per BDM memo < slab thickness**): $t_{s2} = 7.5\,in$

$A_s := \frac{percent}{100} \cdot A_{sp}$ $A_s = 0.28\,in^2$ per ft

use bar # $bar := 4$ $s := 7.5 \cdot in$ $A_s := A_b(bar) \cdot \frac{1 \cdot ft}{s}$ $A_s = 0.32\,in^2$ per ft **OK**

6.8 Maximum bar spacing (§5.10.3.2)

Unless otherwise specified, the spacing of the primary reinforcement in walls and slabs shall not exceed 1.5 times the thickness of the member or 18 in.. The maximum spacing of temperature shrinkage reinforcement shall be as specified in §5.10.8.

$1.5 \cdot t_{s1} = 10.5\,in$ **OK**

6.9 Protective Coating (§5.12.4)

Epoxy coated reinforcement shall be used for slab top layer reinforcements except when the slab is overlaid with asphalt.

7 Slab Overhang Design

(§3.6.1.3.4) Horizontal loads resulting from vehicular collision with barrier shall be considered in accordance with the provisions of LRFD Section 13.
(§13.7.3.1.2) Unless a lesser thickness can be proven satisfactory during the crash testing procedure, the min. edge thickness for concrete deck overhangs shall be taken as 8 in. for concrete deck overhangs supporting concrete parapets or barriers.

7.1 Applicable Limit States (§5.5.1)

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied.

Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck (§13.6.2).

7.2 Strength Limit state

Load Modifier

$\eta_D := 1.00$ for ductile components and connections (§1.3.3 & simplified)

$\eta_R := 1.00$ for redundant members (§1.3.4)

$\eta_I := 1.00$ for operationally important bridge (§1.3.5)

$\eta := \max\left(\begin{pmatrix} \eta_D \cdot \eta_R \cdot \eta_I \\ 0.95 \end{pmatrix}\right)$ $\eta = 1$ (§1.3.2)

Load factors (LRFD Table 3.4.1-1):

$\gamma_p := 1.25$ for component and attachments

$\gamma_L := 1.75$ for LL

7.3 Extreme Event Limit State II

Extreme event limit state shall apply for the force effect transmitted from the vehicular collision force.

Load Modifier

$\eta_D := 1.00$ (§1.3.3)

$\eta_R := 1.00$ (§1.3.4)

$\eta_I := 1.00$ (§1.3.5)

$\eta_e := \max\left(\begin{pmatrix} \eta_D \cdot \eta_R \cdot \eta_I \\ 0.95 \end{pmatrix}\right)$ $\eta_e = 1$ (§1.3.2)

Load factors (LRFD Table 3.4.1-1):

$\gamma_p := 1.25$ for component and attachments

$\gamma_{CT} := 1.00$ for collision force

7.4 Vehicular Collision Force (§13.7.2)

Railing test level TL-4 applies for high-speed highways, freeways, and interstate highways with a mixture of trucks and heavy vehicles.

The transverse and longitudinal loads need not be applied in conjunction with vertical loads (§A13.2). Design forces for railing test level **TL-4** (LRFD Table A13.2-1),

transverse $F_t := 54 \cdot kip$

longitudinal $F_L := 18 \cdot kip$

vertical (down) $F_v := 18 \cdot kip$

Effective Distances:

transverse $L_t := 3.50 \cdot ft$

longitudinal $L_L := 3.50 \cdot ft$

vertical $L_v := 18 \cdot ft$

Min. design height, H, 32 in. (LRFD Table A13.2-1) use $H := 34 \cdot in$

7.5 Design Procedure (§A13.3)

Yield line analysis and strength design for reinforced concrete may be used.

7.6 Nominal Railing Resistance (§A13.3)

For single-slope barriers, the approximate flexural resistance may be taken as:

Flexural capacity about vertical axis,

$$M_w := 18.5 \cdot \frac{kip \cdot ft}{ft}$$

Additional flexural resistance of beam in addition to M_w, if any, at top of wall,

$$M_b := 0.00 \cdot kip \cdot ft$$

Flexural capacity about horizontal axis,

$$M_c := 17.1 \cdot \frac{kip \cdot ft}{ft}$$

Critical wall length, over which the yield mechanism occurs, L_c, shall be taken as:

$$L_c := \frac{L_t}{2} + \sqrt{\left(\frac{L_t}{2}\right)^2 + \frac{8 \cdot H \cdot \left(M_b + M_w \cdot H\right)}{M_c}} \qquad L_c = 10.27\,ft$$

For impact within a barrier segment, the total transverse resistance of the railing may be taken as:

$$R_w := \left(\frac{2}{2 \cdot L_c - L_t}\right) \cdot \left(8 \cdot M_b + 8 \cdot M_w \cdot H + \frac{M_c \cdot {L_c}^2}{H}\right) \qquad R_w = 123.93\,kip$$

7.7 Design Load Cases (§A13.4.1)

Case 1

Transverse and longitudinal forces at extreme event limit state.

Resistance factor (§A13.4.3.2) $\phi := 1.0$

(§C13.7.3.1.2) Presently, in adequately designed bridge deck overhangs, the major crash-related damage occurs in short sections of slab areas where the barriers is hit.

a. at inside face of parapet

$$M_s := \frac{\min((R_w \;\; 1.2 \cdot F_t)) \cdot H}{(L_c + 2 \cdot H)}$$ moment capacity of the base of the parapet (see memo),

$$M_s = 11.5 \frac{kip \cdot ft}{ft}$$

$$M_{DCa} := 0.55 \cdot \frac{kip \cdot ft}{ft}$$ DL M- at edge of curb (see s-DL STRUDL output), $$\frac{cw}{overhang} = 0.287$$

design moment

$$M_u := \eta_e \cdot (\gamma_p \cdot M_{DCa} + \gamma_{CT} \cdot M_s) \qquad M_u = 12.2 \frac{kip \cdot ft}{ft}$$

(§A13.4.2) Deck overhang may be designed to provide a flexural resistance, M_s, which is acting in coincident with tensile force, T (see memo),

$$T := \frac{\min((R_w \;\; 1.2 \cdot F_t)) \cdot ft}{(L_c + 2 \cdot H)} \qquad T = 4.07\,kip$$ per ft

min. "haunch+slab" dimension, $A := t_{s2} + 0.75 \cdot in$ $A = 8.25\,in$

d_s, flexural moment depth at edge of curb,

assume bar # $bar_o := 5$

$$d_s := \left(7 \cdot in + \frac{A - 7 \cdot in}{overhang - 0.5 \cdot b_f} \cdot cw\right) - 2.5 \cdot in - \frac{dia(bar_o)}{2} \qquad d_s = 4.7\,in$$

A_s required for M_u and T,

$$A_s := \frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_s - \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot ft}{0.85 \cdot \phi \cdot f'_c \cdot ft}}\right) + \frac{T}{f_y} \qquad A_s = 0.64\,in^2$$ per ft (1)

Check max. reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where d_e $d_e := d_s$ $d_e = 4.7\,in$

$$c := \frac{A_s \cdot f_y - T}{0.85 \cdot \beta_1 \cdot f'_c \cdot 1 \cdot ft} \qquad c = 1\,in$$

$$\text{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{c}{d_e} = 0.209$$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

b. at design section in the overhang

Design critical section for negative moment and shear shall be at d_c, (§4.6.2.1.6)

$$d_c := \min\left(\left(\frac{b_f}{3} \quad 15 \cdot in\right)\right) \qquad d_c = 8.33\,in$$

from CL of girder (may be too conservative, see training notes)

At the inside face of the parapet, the collision forces are distributed over a distance L_c for the moment and L_c + 2H for the axial force. Similarly, assume the distribution length is increased in a 30 degree angle from the base of the parapet,

Collision moment at design section,

$$M_{se} := \frac{M_s \cdot L_c}{L_c + 2 \cdot 0.577 \cdot (overhang - cw - d_c)} \qquad M_{se} = 9.69\,\frac{kip \cdot ft}{ft}$$

dead load moment @ d_c **from CL of exterior girder** (see s-slab.gts STRUDL output)

$$M_{DCb} := 1.90 \cdot \frac{kip \cdot ft}{ft}$$

design moment

$$M_u := \eta_e \cdot (\gamma_p \cdot M_{DCb} + \gamma_{CT} \cdot M_{se}) \qquad M_u = 12.07\,\frac{kip \cdot ft}{ft}$$

(§A13.4.2) design tensile force, T,

$$T := \frac{\min((R_w \quad 1.2 \cdot F_t)) \cdot ft}{[L_c + 2 \cdot H + 2 \cdot 0.577 \cdot (overhang - cw - d_c)]} \qquad T = 3.63\,kip$$

per ft

d_s, flexural moment depth at design section in the overhang.

$$d_s := A - 2.5 \cdot in - \frac{dia(bar_o)}{2} \qquad d_s = 5.4\,in$$

A_s required for M_u and T,

$$\frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_s - \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot ft}{0.85 \cdot \phi \cdot f'_c \cdot ft}} \right) + \frac{T}{f_y} = 0.53\,in^2$$

per ft (doesn't control) (2)

c. at design section in first span

(see training notes) The total collision moment can be treated as an applied moment at the end of a continuous strip and the ratio of the moment M2/M1 can be calculated for the transverse design strip. As an approximation, it can be taken equal to the ratio of the **moments produced by the parapet self weight** at the centerline of the first and second girder. The collision moment per unit width at the section under consideration can then be determined using the 30º distribution. Dead load at this design section can be determined by interpolation between dead moments at Centerline of girder and at 0.1S.

Collision moment at exterior girder, $M_s = 11.52 \frac{kip \cdot ft}{ft}$

Collision moment at first interior girder (see s-slab.gtb output, barrier loading only),

$$M_{2i} := M_s \cdot \frac{0.34}{1.68} \qquad M_{2i} = 2.33 \frac{kip \cdot ft}{ft}$$

By interpolation for a section in the first bay at d_c from the exterior girder,

$$M_{si} := M_s - d_c \cdot \frac{M_s + M_{2i}}{S} \qquad M_{si} = 10.1 \frac{kip \cdot ft}{ft}$$

Using the 30º angle distribution, design moment

$$M_{si} := \frac{M_{si} \cdot L_c}{L_c + 2 \cdot 0.577 \cdot (overhang - cw + d_c)} \qquad M_{si} = 7.51 \frac{kip \cdot ft}{ft}$$

dead load moment @ this section (see s-dl.gtb output) $\frac{d_c}{S} = 0.103$

$$M_{DCi} := 2.0 \cdot \frac{kip \cdot ft}{ft}$$

design moment

$$M_u := \eta_e \cdot (\gamma_p \cdot M_{DCi} + \gamma_{CT} \cdot M_{si}) \qquad M_u = 10.01 \frac{kip \cdot ft}{ft}$$

d_s, flexural moment depth at the design section,

$$d_s := t_{s1} - 2.0 \cdot in - \frac{dia(bar_o)}{2} \qquad d_s = 4.69\,in$$

A_s required for M_u,

$$\frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_s - \sqrt{d_s^2 - \frac{2 \cdot M_u \cdot ft}{0.85 \cdot \phi \cdot f'_c \cdot ft}} \right) = 0.46\,in^2$$

per ft (doesn't control) (3)

Case 2 Vertical collision force

For concrete parapets, the case of vertical collision never controls.

Case 3 Check DL + LL

Resistance factor (§1.3.2.1) $\phi_f := 0.9$

For deck overhangs, where applicable, the §3.6.1.3.4 may be used in lieu of the equivalent strip method (§4.6.2.1.3).

a. at design section in the overhang

moment arm for 1.0 kip/ft live load (§3.6.1.3.4)

$$x := \text{overhang} - cw - 1 \cdot ft - d_c \qquad x = 8.17\,in$$

live load moment without impact,

$$w_L := 1.0 \cdot \frac{kip}{ft}$$

$$M_{LL} := M_1 \cdot w_L \cdot x \qquad M_{LL} = 0.82\,\frac{kip \cdot ft}{ft}$$

factored moment

$$M_u := \eta \cdot \left[\gamma_p \cdot M_{DCb} + \gamma_L \cdot M_{LL} \cdot (1.0 + IM) \right] \qquad M_u = 4.28\,\frac{kip \cdot ft}{ft}$$

d_s, flexural moment depth at edge of curb,

$$d_s := A - 2.5 \cdot in - \frac{dia\left(bar_o\right)}{2} \qquad d_s = 5.44\,in$$

A_s required for M_u,

$$\frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_s - \sqrt{d_s^{\,2} - \frac{2 \cdot M_u \cdot ft}{0.85 \cdot \phi_f \cdot f'_c \cdot ft}} \right) = 0.18\,in^2$$ per ft (doesn't control) (4)

b. at design section in first span

Assume slab thickness at this section, $t_{s1} = 7\,in$

use the same D.L. + L.L moment as in previous for design (approximately)

factored moment $M_u = 4.28\,\frac{kip \cdot ft}{ft}$

d_s, flexural moment depth at edge of curb,

$$d_s := t_{s1} - 2.0 \cdot in - \frac{dia(bar_o)}{2} \qquad d_s = 4.7\,in$$

A_s required for M_u,

$$\frac{0.85 \cdot f'_c \cdot ft}{f_y} \cdot \left(d_s - \sqrt{d_s^{\,2} - \frac{2 \cdot M_u \cdot ft}{0.85 \cdot \phi_f \cdot f'_c \cdot ft}} \right) = 0.21\,in^2$$ (doesn't control) (5)

The largest of (1) to (5), As required, $A_s = 0.64\,in^2$ per ft

use bar # $bar_o = 5$ @ $s := 18 \cdot in$ **(top transverse) at edge of curb, bundle 1 #5 to every other top bar in the deck overhang region**

$bar_n = 5$ $s_n = 9\,in$

$$A_s := A_b(bar_o) \cdot \frac{1 \cdot ft}{s} + A_b(bar_n) \cdot \frac{1 \cdot ft}{s_n} \qquad A_s = 0.62\,in^2$$ **say OK**

Check max. reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where **min. d_e** $d_e := t_{s1} - 2.0 \cdot in - \frac{dia(bar_n)}{2}$ $d_e = 4.7\,in$

$$c := \frac{A_s \cdot f_y}{0.85 \cdot \beta_1 \cdot f'_c \cdot 1 \cdot ft} \qquad c = 1.07\,in$$

$$\text{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{c}{d_e} = 0.229$$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

Determine the point in the first bay of the deck where the additional bars are no longer needed,

$$A_s := A_b(bar_n) \cdot \frac{1 \cdot ft}{s_n} \qquad A_s = 0.41\,in^2$$

$$c := \frac{A_s \cdot f_y}{0.85 \cdot \beta_1 \cdot f'_c \cdot 1 \cdot ft} \qquad c = 0.7\,in$$

$$d_e := t_{s1} - 2.0 \cdot in - \frac{dia(bar_n)}{2} \qquad d_e = 4.7\,in$$

$$a := \beta_1 \cdot c \qquad a = 0.6\,in$$

For the strength limit state,

$$M_{cap} := \phi_f \cdot A_s \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \qquad M_{cap} = 8.15\,kip \cdot ft \qquad \text{per ft}$$

For the extreme event limit state,

$$M_{cap} := \phi \cdot A_s \cdot f_y \cdot \left(d_e - \frac{a}{2}\right) \qquad M_{cap} = 9.06\,kip \cdot ft \qquad \text{per ft}$$

By inspection of (1) to (5), no additional bar is required beyond design section of the first bay.

Cut off length requirement (§5.11.1.2)

$15 \cdot dia\left(bar_o\right) = 0.781\,ft$ (controls by inspection)

8 Reinforcing Details

8.1 Development of Reinforcement (§5.11.2.1.1)

basic development length for #11 bar and smaller,

$$L_{db}\left(d_b, A_b\right) := \max\left(\begin{pmatrix} 1.0 \cdot ft \\ \dfrac{1.25 \cdot A_b \cdot f_y \cdot \sqrt{ksi}}{in \cdot ksi \cdot \sqrt{f'_c}} \\ 0.4 \cdot d_b \cdot \dfrac{f_y}{ksi} \end{pmatrix}\right)$$

For **#5** bars, $L_{db}\left(0.625 \cdot in, 0.31 \cdot in^2\right) = 15\,in$

For **#6** bars, $L_{db}\left(0.75 \cdot in, 0.44 \cdot in^2\right) = 18\,in$

For **epoxy coated** bars (§5.11.2.1.2),

with cover less than 3d_b or with clear spacing less than 6d_b ....times 1.5

not covered above ......times 1.2

For **widely spaced** bars..... times 0.8 (§5.11.2.1.3)

bars spaced laterally not less than 150 mm center-to-center, with not less than 75 mm clear cover measured in the direction of spacing.

For **bundled** bars..... times 1.2 for a three-bar bundle (§5.11.2.3)

Lap Splices in Tension (§5.11.5.3.1)

The length of lap for tension lap splices shall not be less than either 300 mm or the following for

Class A, B, or C splices:

Class A splice .... times 1.0
Class B splice .... times 1.3
Class C splice .... times 1.7

Flexural Reinforcement (§5.11.1.2)

Except at supports of simple-spans and at the free ends of cantilevers, reinforcement shall be extended beyond the point at which it is no longer required to resist flexure for a distance not less than:

the effective depth of the member,
15 times the nominal diameter of bar, or
1/20 of the clear span.

No more than 50% of the reinforcement shall be terminated at any section, and adjacent bars shall not be terminated in the same section.

Positive moment reinforcement (§5.11.1.2.2)

At least 1/3 the positive moment reinforcement in simple-span members, and 1/4 the positive moment reinforcement in continuous members, shall extend along the same face of the member beyond the centerline of the support. In beams, such extension shall not be less than 150 mm.

Negative moment reinforcement (§5.11.1.2.3)

At least 1/3 of the total tension reinforcement provided for negative moment at a support shall have an embedment length length beyond the point of inflection (DL + LL) not less than:

the effective depth of the member, d
12.0 d_b, and
0.0625 times the clear span.

Moment resisting joints (§5.11.1.2.4)

In Seismic Zones 3 and 4, joint shall be detailed to resist moments and shears resulting from horizontal loads through the joint.

Q.E.D.

Design Example 3 Precast Slab Design Stay-In-Place (SIP) Deck Panel

Design Criteria

Loading: HL-93

Concrete:

SIP Panel, $f'_{ci} := 4.0 \cdot ksi$

$f'_{c} := 5.0 \cdot ksi$ (f'ci + 1 ksi)

CIP slab, $f'_{cs} := 4.0 \cdot ksi$

Reinforcing Steel: (§5.4.3)

AASHTO M-31, Grade 60, $f_y := 60 \cdot ksi$ $E_s := 29000 \cdot ksi$

Prestressing Steel:

AASHTO M-203, uncoated 7 wire, low-relaxation strands (§5.4.4.1)

Nominal strand diameter, $db := 0.375 \cdot in$ $Ap := 0.085 \cdot in2$

(Trends now are toward the use of 3/8in. diameter strand, per PCI J., 33(2), pp.67-109)

$f_{pu} := 270 \cdot ksi$

$f_{py} := 0.90 \cdot f_{pu}$ $f_{py} = 243\ ksi$

$f_{pe} := 0.80 \cdot f_{py}$ $f_{pe} = 194.4\ ksi$ @ service limit state after all losses (LRFD Table 5.9.1-1)

$E_p := 28500 \cdot ksi$

Design Method: LRFD 2nd Edition

Mechanical shear ties on the top of panels are not required per PCI, special report, PCI J., 32(2), pp. 26-45.

Structure:

Design span	$L := 89.07 \cdot ft$		
Roadway width	BW 53.0·ft	barrier face to barrier face	
Girder spacing	$S := 6.75 \cdot ft$		
Skew angle	$\theta := 14.65 \cdot deg$		
no. of girder	$N_b := 8$		
curb width on deck,	$cw := 10.5 \cdot in$		
Deck overhang (CL. of exterior girder to end of deck)	$overhang := \frac{BW - (N_b - 1) \cdot S}{2} + cw$	$overhang = 3.75ft$	
slab design thickness	$t_s1 := 8.0 \cdot in$		
for D.L. calculation	$t_s2 := 8.5 \cdot in$		
Panel dimensions:	$W_{sip} := 8.0 \cdot ft$	$L_{sip} := 6.34 \cdot ft$	$t_{sip} := 3.5 \cdot in$
CIP composite slab:	$t_{cs}1 := t_s1 - t_{sip}$	$t_{cs}1 = 4.5in$	(used for structural design)
	$t_{cs}2 := t_s2 - t_{sip}$	$t_{cs}2 = 5in$	(actual thickness)
	$w_c := 0.160 \cdot kcf$		
future wearing surface	$t_{ws} := 0 \cdot in$		

Minimum Depth and Cover (§9.7.1)

Min. Depth $\quad \text{if}(t_{s2} \geq 7.0 \cdot in, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

Min. SIP thickness $\quad \text{if}(0.55 \cdot t_{s2} > t_{sip} \geq 3.5 \cdot in, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

top cover for epoxy-coated main reinforcing steel
= 1.5in. (up to #11 bar)
= 2.0in. (#14 & #18 bars) (§5.12.4 & Table 5.12.3-1)

sacrificial thickness = 0.5in. (§2.5.2.4)

Opitional deflection criteria for span-to-depth ratio (LRFD Table 2.5.2.6.3.1)

Min. Depth (continuous span) where S = 6.75ft (slab span length):

$$\text{if}\left[\max\left(\begin{pmatrix} \dfrac{S + 10 \cdot ft}{30} \\ 0.54 \cdot ft \end{pmatrix}\right) \leq t_{s1}, \text{"OK"}, \text{"NG"}\right] = \text{"OK"} \qquad \max\left(\begin{pmatrix} \dfrac{S + 10 \cdot ft}{30} \\ 0.54 \cdot ft \end{pmatrix}\right) = 6.7in$$

Skew Deck (§9.7.1.3)

$\theta \leq 25 \cdot deg = 1$ it true, the primary reinforcement may be placed in the direction of the skew; otherwise, it shall be placed perpendicular to the main supporting components.

Loads

The precast SIP panels support their own weight, any construction loads, and the weight of the CIP slabs. For superimposed dead and live loads, the precast panels are analyzed assuming that they act compositely with the CIP concrete.

Dead load per foot

SIP panel $\quad w_{sip} := t_{sip} \cdot w_c \qquad w_{sip} = 0.047 \dfrac{kip}{ft^2}$

CIP slab $\quad w_{cs} := t_{cs2} \cdot w_c \qquad w_{cs} = 0.067 \dfrac{kip}{ft^2}$

Weigth of one traffic barrier is $\quad tb := 0.52 \cdot \dfrac{kip}{ft^2}$

Weigth of one sidewalk is $\quad tside := 0.52 \cdot \dfrac{kip}{ft^2}$

Wearing surface & construction loads

wearing surface $\quad w_{ws} := t_{ws} \cdot w_c \cdot ft \qquad w_{ws} = 0 \dfrac{kip}{ft}$

construction load $\quad w_{con} := 0.050 \cdot \dfrac{kip}{ft}$ (§9.7.4.1)
(applied to deck panel only)

Live Loads

(§3.6.1.3, not for empircial design mthod) Where deck is designed using the approximate strip method, specified in §4.6.2.1, the live load shall be taken as the wheel load of the 32.0kip axle of the design truck, without lane load, where the strips are transverse.

$\text{if}(S \leq 15 \cdot ft, \text{"OK"}, \text{"NG"}) = \text{"OK"}$ (§3.6.1.1.1.2)

Multiple presence factor: $\quad M1 := 1.2 \qquad M2 := 1.0$ (§3.6.1.1.1.2)

Dynamic Load Allowance (impact) $\quad IM := 0.33$ (§3.6.2.1)

Maximum factored moments **per unit width** based on Table A4-1: for S = 2.057m
(include multiple presence factors and the dynamic load allowance)

applicability if [min((0.625·S 6·ft)) ≥ overhang - cw, "OK", "NG"] = "OK"
if (N_b ≥ 3, "OK", "NG") = "OK"

$$M_{LLp} := 5.10 \cdot \frac{kip \cdot ft}{ft}$$

(§3.6.1.3.4) For deck overhang design with a cantilever, not exceeding 6.0ft from the centerline of the exterior girder to the face of a continuous concrete railing, the wheel loads may be replaced with a uniformly distributed line load of 1.0 KLF intensity, located 1ft from the face of the railing.

if (overhang - cw ≤ 6·ft, "OK", "NG") = "OK"

Load combination

Where traditional design based on flexure is used, the requirements for strength and service limit states shall be satisfied.

Extreme event limit state shall apply for the force effect transmitted from the bridge railing to bridge deck (§13.6.2).

Fatigue need not be investigated for concrete deck slabs in multi-girder applications (§5.5.3.1).

Strength Limit States

Load Modifier

$\eta D := 1.00$ for conventional design (§1.3.3)

$\eta R := 1.00$ for conventional level of redundancy (§1.3.4)

$\eta I := 1.00$ for typical bridges (§1.3.5)

$$\eta := \max\left(\left(\frac{\eta D \cdot \eta R \cdot \eta I}{0.95}\right)\right) \qquad \eta = 1 \qquad (§1.3.2)$$

Strength I load combination - normal vehicular load without wind (§3.4.1)

Load factors (LRFD Table 3.4.1-1 & 2):

$\gamma_p := 1.25$ for component and attachments

$\gamma DW := 1.50$ for DW

$\gamma L := 1.75$ for LL

Section Properties

Non-composite section per foot

$A_{sip} := t_{sip} \cdot 12 \cdot in$ $\qquad A_{sip} = 42\,in^2$

$$I_{sip} := \frac{12 \cdot in \cdot t_{sip}^3}{12} \qquad I_{sip} = 42.875\,in^4$$

$$Y_{bp} := \frac{t_{sip}}{2} \qquad Y_{bp} = 1.75\,in$$

$$Y_{tp} := t_{sip} - Y_{bp} \qquad S_{tp} := \frac{I_{sip}}{Y_{tp}} \qquad S_{bp} := \frac{I_{sip}}{Y_{bp}}$$

$$Y_{tp} = 1.75\,in \qquad S_{tp} = 24.5\,in^3 \qquad S_{bp} = 24.5\,in^3$$

$$E_c := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \qquad E_c = 4.723 \times 10^3\, ksi \qquad (§5.4.2.4)$$

$$E_{ci} := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi \qquad E_{ci} = 4.224 \times 10^3\, ksi$$

Composite Section Properties (§4.6.2.6)

$$E_{cs} := 33000 \cdot \left(\frac{w_c}{kcf}\right)^{1.5} \cdot \sqrt{\frac{f'_{cs}}{ksi}} \cdot ksi \qquad E_{cs} = 4.224 \times 10^3\, ksi \qquad (§5.4.2.4)$$

modular ratio, $n := \sqrt{\frac{f'_c}{f'_{cs}}}$ $\qquad n = 1.118$

$b := 12 \cdot in$

$$A_{slab} := \frac{b}{n} \cdot t_{cs1} \qquad Y_{bs} := t_{sip} + \frac{t_{cs1}}{2} \qquad AY_{bs} := A_{slab} \cdot Y_{bs}$$

	Area	Y_b	$A \cdot Y_b$
CIP slab	$A_{slab} = 48.3\, in^2$	$Y_{bs} = 5.75\, in$	$A_{slab} \cdot Y_{bs} = 277.7\, in^3$
SIP panel	$A_{sip} = 42\, in^2$	$Y_{bp} = 1.75\, in$	$A_{sip} \cdot Y_{bp} = 73.5\, in^3$

$$Y_b := \frac{A_{slab} \cdot Y_{bs} + A_{sip} \cdot Y_{bp}}{A_{slab} + A_{sip}} \qquad Y_b = 3.89\, in \qquad \text{@ bottom of panel}$$

$$Y_t := t_{sip} - Y_b \qquad Y_t = -0.39\, in \qquad \text{@ top of panel}$$

$$Y_{ts} := t_{sip} + t_{cs1} - Y_b \qquad Y_{ts} = 4.11\, in \qquad \text{@ top of slab}$$

$$I_{slabc} := A_{slab} \cdot \left(Y_{ts} - \frac{t_{cs1}}{2}\right)^2 + \frac{\left(\frac{b}{n}\right) \cdot t_{cs1}^{\,3}}{12} \qquad I_{slabc} = 248.687\, in^4$$

$$I_{pc} := A_{sip} \cdot \left(Y_b - Y_{bp}\right)^2 + I_{sip} \qquad I_{pc} = 235.131\, in^4$$

$$I_c := I_{slabc} + I_{pc} \qquad I_c = 483.818\, in^4$$

Section modulous of the composite section

$$S_b := \frac{I_c}{Y_b} \qquad S_b = 124.39\, in^3 \qquad \text{@ bottom of panel}$$

$$S_t := \frac{I_c}{|Y_t|} \qquad S_t = 1.242 \times 10^3\, in^3 \qquad \text{@ top of panel}$$

$$S_{ts} := n \cdot \frac{I_c}{Y_{ts}} \qquad S_{ts} = 131.6\, in^3 \qquad \text{@ top of slab}$$

Required Prestress

Assume the span length conservatively as the panel length, $L_{sip} = 1.932\,m$

$$M_{sip} := \frac{w_{sip} \cdot L_{sip}^{2}}{8} \qquad M_{sip} = 0.234\,\frac{ft \cdot kip}{ft}$$

$$M_{cip} := \frac{w_{cs} \cdot L_{sip}^{2}}{8} \qquad M_{cip} = 0.335\,\frac{ft \cdot kip}{ft}$$

For the superimposed dead and live loads, the force effects should be calculated based on analyzing the strip as a continuous beam supported by infinitely rigid supports (§4.6.2.1.6)

$$M_{ws} := 0 \cdot \frac{ft \cdot kip}{ft}$$

$$M_{sidl} := 0.19 \cdot \frac{kip \cdot ft}{ft}$$ (see Strudl s-dl output)

$$f_b := \frac{(M_{sip} + M_{cip})\,ft}{S_{bp}} + \frac{(M_{ws} + M_{sidl} + M_{LLp}) \cdot ft}{S_b} \qquad f_b = 0.789\,ksi$$

Tensile Stress Limits

$$0.190 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 0.42\,ksi$$ (§5.9.4.2.2)

$0 \cdot ksi$ WSDOT design practice

Required precompression stress at bottom fiber,

$$f_{creq} := f_b - 0 \cdot ksi \qquad f_{creq} = 0.789\,ksi$$

If P_{se} is the total effective prestress force after all losses, and the center of gravity of stands is concentric with the center of gravity of the SIP panel:

$$P_{se} := f_{creq} \cdot W_{sip} \cdot t_{sip} \qquad P_{se} = 265.184\,kip$$ per panel

Assume stress at transfer,

$$f_{pi} := 0.75 \cdot f_{pu} \qquad f_{pi} = 202.5\,ksi$$ (LRFD Table 5.9.3-1)

Assume 15% final losses, the final effective prestress,

$$p_{se} := f_{pi} \cdot (1 - 0.15) \qquad p_{se} = 172.125\,ksi$$

The required number of strands, $N_{req} := \dfrac{P_{se}}{p_{se} \cdot A_p}$ $\quad N_{req} = 18.125 \quad N_p := \mathrm{ceil}(N_{req})$

Try $N_p := 19$

Prestress Losses

Loss of Prestress (§5.9.5)

$$\Delta f_{pT} = \Delta f_{pES} + \Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$$

steel relaxation at transfer (§5.9.5.4.4b)

Curing time for concrete to attain f'ci is approximately 12 hours: set $t := 0.75$ day

Guess values: $\Delta f_{pR1} := 2.0 \cdot ksi$ $f_{pj} := 205 \cdot ksi$

Given $\Delta f_{pR1} = \dfrac{\log(24.0 \cdot t)}{40.0} \cdot \left(\dfrac{f_{pj}}{f_{py}} - 0.55\right) \cdot f_{pj}$

$f_{pj} = 0.75 \cdot f_{pu} + \Delta f_{pR1}$ immediately prior to transfer+steel relax. (LRFD Table 5.9.3-1)

$\begin{pmatrix} f_{pj} \\ \Delta f_{pR1} \end{pmatrix} := \mathrm{Find}(f_{pj}, \Delta f_{pR1})$ $f_{pj} = 204.4\,ksi$

$\Delta f_{pR1} = 1.87\,ksi$

Given: $A_p = 0.085\,in^2$

straight strands $N_p = 19$ jacking force, $f_{pj} \cdot N_p \cdot A_p = 330.05\,kip$

(note: these forces include initial prestress relaxation loss, see §C5.9.5.4.4b)

$A_{ps} := A_p \cdot N_p$ $A_{ps} = 1.615\,in^2$ per panel

$A_{psip} := A_{ps} \cdot \dfrac{ft}{W_{sip}}$ $A_{psip} = 0.202\,in^2$ per ft

c.g. of all strands to c.g. of girder, $e_p := 0 \cdot in$

Elastic Shortening, Δf_{pES} (§5.9.5.2.3a)

$f_{cgp:}$ concrete stress at c.g. of prestressing tendons due to the prestressing force at transfer and the self-weight of the member at the sections of maximum moment.

Guess values: $p_{si} := 196.2 \cdot ksi$ prestress tendon stress at transfer (LRFD Table 5.9.3-1)

Given $\left(f_{pj} - \Delta f_{pR1} - p_{si}\right) \cdot \dfrac{E_{ci}}{E_p} = -\left[\dfrac{-(p_{si} \cdot A_{psip})}{A_{sip}}\right]$ (note: used only when e_p = 0 in)

$p_{si} := \mathrm{Find}(p_{si})$ $\quad\quad p_{si} = 196.1\,\mathrm{ksi}$

$$f_{cgp} := \frac{-[p_{si}\cdot(A_{psip})]}{A_{sip}} \quad\quad f_{cgp} = -0.94\,\mathrm{ksi}$$

$\Delta f_{pES} := f_{pj} - \Delta f_{pR1} - p_{si}$ $\quad\quad \Delta f_{pES} = 6.36\,\mathrm{ksi}$

9.3.2 Approximate Lump Sum Estimate of Time Dependent Losses (§5.9.5.3)

Time-dependent losses : $\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR2}$

Criteria: $\mathrm{if}(f'_{ci} > 3.5\mathrm{ksi}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$

Normal density concrete
Concrete is either steam or moist cured
Prestressing is by low relaxation strands
Are sited in average exposure condition and temperatures

no partial prestressing $A_s := 0\cdot \mathrm{in}^2$

partial prestress ratio (LRFD Eq. 5.5.4.2.1-2) $PPR := \dfrac{A_{ps}\cdot f_{py}}{A_{ps}\cdot f_{py} + A_s\cdot f_y}$ $\quad\quad PPR = 1$

Approximate lump sum estimate of time-dependent losses (§5.9.5.3)

For solid slab,

$LOSS_t := 29.0\cdot \mathrm{ksi} + 4.0\cdot \mathrm{ksi}\cdot PPR$ (upper bound) $\quad\quad LOSS_t = 33\,\mathrm{ksi}$

Allowable reduction for solid slab, 6.0 ksi,

$LOSS_t := LOSS_t - 6.0\cdot \mathrm{ksi}$ $\quad\quad LOSS_t = 27\,\mathrm{ksi}$

(§5.9.5.1) In pretension members where the approximate lump sum estimate of losses is used, Δf_{pR1} should be deducted from the total relaxation.

At transfer, the losses that could be accounted for are elastic shortening and steel relaxation only.

$LOSS_t := LOSS_t - \Delta f_{pR1}$ $\quad\quad LOSS_t = 25.13\,\mathrm{ksi}$

Loss due to Creep Δf_{pCR} ***(§5.9.5.2.3a)***

Δf_{cdp}, change in concrete stress at center of gravity of prestressing steel due to permanent loads, except the load acting at the time the prestressing force is applied. Values of Δf_{cdp} should be calculated at the same section or sections for which f_{cgp} is calculated.

$M_{cip} = 0.335\,\dfrac{\mathrm{ft}\cdot\mathrm{kip}}{\mathrm{ft}}$ (act on the non-composite section)

$M_{ws} = 0 \frac{ft \cdot kip}{ft}$ (act on the composite section)

$M_{sidl} = 0.19 \frac{ft \cdot kip}{ft}$ (act on the composite section)

However, the weight of the CIP slab provides zero stress at the center of gravity of pretensioning force.
So, stresses due only to wearing surface and barriers are considered.

c.g. of all strands to c.g. of composite girder,

$e_{pc} := Y_b - \frac{t_{sip}}{2}$ $e_{pc} = 2.14\,in$

$\Delta f_{cdp} := \frac{(M_{sidl} + M_{ws}) \cdot ft \cdot e_{pc}}{I_c}$ $\Delta f_{cdp} = 0.01\,ksi$

$\Delta f_{pCR} := 12.0 \cdot (-f_{cgp}) - 7.0 \cdot \Delta f_{cdp}$ $\Delta f_{pCR} = 11.24\,ksi$

Total loss Δf_{pT} (note: BDM assumes a 330 MPa total loss), not including Δf_{pR1},

$\Delta f_{pT} := LOSS_t + \Delta f_{pES}$ $\Delta f_{pT} = 31.49\,ksi$

$f_{pe} := f_{pj} - \Delta f_{pRl} - \Delta f_{pT}$ $f_{pe} = 171.005\,ksi$

$\text{if}\left(f_{pe} \le 0.80 \cdot f_{py}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$ (LRFD Table 5.9.3-1)

$P_e := \frac{N_p \cdot A_p \cdot f_{pe}}{W_{sip}}$ $P_e = 34.522 \frac{kip}{ft}$ per foot

Stresses in the SIP Panel at Transfer

Stress Limits for Concrete

Compression: $-0.60 \cdot f'_{ci} = -2.4\,ksi$

Tension: Allowable tension with bonded reinforcement which is sufficient to resist 120% of the tension force in the cracked concrete computed on the basis of an uncracked section (§5.9.4.1.2).

$$0.24 \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi = 0.48\,ksi$$

or w/o bonded reinforcement,

$$\min\left(\begin{pmatrix} 0.0948 \cdot \sqrt{\frac{f'_{ci}}{ksi}} \cdot ksi \\ 0.200 \cdot ksi \end{pmatrix}\right) = 0.19\,ksi \qquad \text{(Controls)}$$

Because the strand group is concentric with the precast concrete panel, the midspan section is the critical section that should be checked.

Stress at Midspan

Effective stress after transfer,

$$P_{si} := \frac{N_p \cdot A_p \cdot p_{si}}{W_{sip}} \qquad P_{si} = 39.596 \frac{kip}{ft}$$

Moment due to weight of the panel,

$$M_{sip} = 0.234 \frac{kip \cdot ft}{ft}$$

At top of the SIP panel,

$$\left(-\frac{M_{sip} \cdot ft}{S_{tp}} - \frac{P_{si} \cdot ft}{A_{sip}}\right) = -1.06\,ksi \qquad < \text{ allowable } \quad -0.60 \cdot f'_{ci} = -2.4\,ksi \qquad \textbf{OK}$$

At bottom of the SIP panel,

$$\left(\frac{M_{sip} \cdot ft}{S_{bp}} - \frac{P_{si} \cdot ft}{A_{sip}}\right) = -0.83\,ksi \qquad < \text{ allowable } \quad -0.60 \cdot f'_{ci} = -2.4\,ksi \qquad \textbf{OK}$$

Stresses in SIP Panel at Time of Casting Topping Slab

The total prestress after all losses,

$$P_e = 34.522 \frac{kip}{ft}$$

Stress Limits for Concrete

Flexural stresses due to unfactored construction loads shall not exceed 65% of the 28-day compressive strength for concrete in compression or the modulus of rupture in tension for prestressed concrete form panels (§9.7.4.1).

The construction load shall be taken to be less than the weight of the form and the concrete slab plus 0.050 KSF.
For load combination Service I:

Compression: $-0.65 \cdot f'_c = -3.25\,ksi$

Tension: Modulous of rupture,

$$f_r := 0.24 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi \qquad f_r = 0.537\,ksi$$

Stresses at Midspan after all Non-Composite Loads

$$M_{sip} = 0.234 \frac{ft \cdot kip}{ft}$$

$$M_{cip} = 0.335 \frac{ft \cdot kip}{ft}$$

$$M_{const} := 0.050 \cdot \frac{kip}{ft^2} \frac{L_{sip}^2}{8} \qquad M_{const} = 0.251 \frac{ft \cdot kip}{ft}$$

At top of the SIP panel,

$$\left[-\frac{(M_{sip} + M_{cip} + M_{const}) \cdot ft}{S_{tp}} - \frac{P_e \cdot ft}{A_{sip}} \right] = -1.22\,ksi \quad < \text{ allowable } \quad -0.65 \cdot f'_c = -3.25\,ksi \qquad \textbf{OK}$$

At bottom of the SIP panel,

$$\left[\frac{(M_{sip} + M_{cip} + M_{const}) \cdot ft}{S_{bp}} - \frac{P_e \cdot ft}{A_{sip}} \right] = -0.42\,ksi \quad < \text{ allowable } \quad -0.65 \cdot f'_c = -3.25\,ksi \qquad \textbf{OK}$$

Elastic Deformation (§9.7.4.1)

Deformation due to

$$\Delta := \frac{5}{48} \cdot \frac{(M_{sip} + M_{cip}) \cdot ft \cdot L_{sip}^2}{E_c \cdot I_{sip}} \qquad \Delta = 0.02\,in$$

$$\text{if}\left[\Delta \le \begin{vmatrix} \min\left(\left(\frac{L_{sip}}{180} \quad 0.25 \cdot in\right)\right) & \text{if } L_{sip} \le 10 \cdot ft \\ \min\left(\left(\frac{L_{sip}}{240} \quad 0.75 \cdot in\right)\right) & \text{otherwise} \end{vmatrix}, \text{"OK"}, \text{"NG"} \right] = \text{"OK"}$$

Stresses in SIP Panel at Service Loads

Compression:

- Stresses due to permanent loads

$-0.45 \cdot f'_c = -2.25\,ksi$ for SIP panel

$-0.45 \cdot f'_{cs} = -1.8\,ksi$ for CIP panel

- Stresses due to permanent and transient loads

$-0.60 \cdot f'_c = -3\,ksi$ for SIP panel

$-0.60 \cdot f'_{cs} = -2.4\,ksi$ for CIP panel

- Stresses due to live load + one-half of the permanent loads

  $-0.40 \cdot f'_c = -2\,\text{ksi}$ for SIP panel

  $-0.40 \cdot f'_{cs} = -1.6\,\text{ksi}$ for CIP panel

Tension:

$0.0948 \cdot \sqrt{\frac{f'_c}{\text{ksi}}} \cdot \text{ksi} = 0.21\,\text{ksi}$ (§5.9.4.2.2)

$0 \cdot \text{ksi}$ WSDOT design practice

Service Load Stresses at Midspan

- *Compressive stresses at top of CIP slab*

  Stresses due to permanent load + prestressing

$$-\frac{(M_{ws} + M_{sidl}) \cdot \text{ft}}{S_{ts}} = -0.017\,\text{ksi} \quad < \text{ allowable } \quad -0.45 \cdot f'_{cs} = -1.8\,\text{ksi} \quad \textbf{OK}$$

  Stresses due to permanent and transient loads,

$$-\frac{(M_{ws} + M_{sidl} + M_{LLp}) \cdot \text{ft}}{S_{ts}} = -0.48\,\text{ksi} \quad < \text{allowable} \quad -0.60 \cdot f'_{cs} = -2.4\,\text{ksi} \quad \textbf{OK}$$

- *Compressive stresses at top of the SIP panel*

  Stresses due to permanent load + prestressing

$$-\left(\frac{P_e \cdot \text{ft}}{A_{sip}}\right) - \frac{(M_{sip} + M_{cip}) \cdot \text{ft}}{S_{tp}} - \frac{(M_{ws} + M_{sidl}) \cdot \text{ft}}{S_t} = -1.1\,\text{ksi} \quad < \text{ allowable } \quad -0.45 \cdot f'_c = -2.25\,\text{ksi} \quad \textbf{OK}$$

  Stresses due to permanent and transient loads,

$$-\left(\frac{P_e \cdot \text{ft}}{A_{sip}}\right) - \frac{(M_{sip} + M_{cip}) \cdot \text{ft}}{S_{tp}} - \frac{(M_{ws} + M_{sidl} + M_{LLp}) \cdot \text{ft}}{S_t} = -1.15\,\text{ksi} \quad < \text{allowable} \quad -0.60 \cdot f'_c = -3\,\text{ksi} \quad \textbf{OK}$$

Stresses due to live load + one-half the sum of effective prestress and permanent loads,

$$-0.5 \cdot \left(\frac{P_e \cdot \text{ft}}{A_{sip}}\right) - \frac{0.5 \cdot (M_{sip} + M_{cip}) \cdot \text{ft}}{S_{tp}} - \frac{(0.5 \cdot M_{ws} + 0.5 \cdot M_{sidl} + M_{LLp}) \cdot \text{ft}}{S_t} = -0.6\,\text{ksi}$$

$$< \text{allowable} \quad -0.40 \cdot f'_c = -2\,\text{ksi} \quad \textbf{OK}$$

- *Tensile stresses at bottom of the SIP panel*

  Stresses due to permanent and transient loads,

$$-\left(\frac{P_e \cdot ft}{A_{sip}}\right) + \frac{(M_{sip} + M_{cip}) \cdot ft}{S_{bp}} + \frac{(M_{ws} + M_{sidl} + M_{LLp}) \cdot ft}{S_b} = -0.03\,ksi$$

< allowable $\quad 0.0948 \cdot \sqrt{\frac{f'_c}{ksi}} \cdot ksi = 0.21\,ksi$ **OK**

$0 \cdot ksi$ (BDM)

9.8.11 Flexural Strength of Positive Moment Section

Resistance factors (§5.5.4.2.1)

$\phi_f := 0.90$ for flexure and tension of reinforced concrete

$\phi_p := 1.00$ for flexure and tension of prestressed concrete

$\phi_v := 0.90$ for shear and torsion

Ultimate Moment Required for Strength I

Dead load moment,

$M_{DC} := M_{sip} + M_{cip} + M_{sidl}$ $\qquad M_{DC} = 0.759\,\frac{kip \cdot ft}{ft}$

Wearing surface load moment,

$M_{ws} = 0\,\frac{kip \cdot ft}{ft}$

Live load moment,

$M_{LLp} = 5.1\,\frac{kip \cdot ft}{ft}$

$M_u := \eta \cdot (\gamma_p \cdot M_{DC} + \gamma_{DW} \cdot M_{ws} + \gamma_L \cdot M_{LLp})$ $\qquad M_u = 9.874\,\frac{kip \cdot ft}{ft}$

Flexural Resistance (§5.7.3)

Find stress in prestressing steel at nominal flexural resistance, f_{ps} (§5.7.3.1.1)

$f_{pe} = 171.005\,ksi$ $\qquad 0.5 \cdot f_{pu} = 135\,ksi$

$\text{if}\left(f_{pe} \ge 0.5 \cdot f_{pu}, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$k := 2 \cdot \left(1.04 - \frac{f_{py}}{f_{pu}}\right)$ $\qquad k = 0.28$ $\qquad$ (LRFD Eq. 5.7.3.1.1-2)

$A_s := 0 \cdot in^2$

$A'_s := 0 \cdot in^2$ (conservatively)

d_p, distance from extreme compression fiber to the centroid of the prestressing tendons,

$$d_p := t_{s1} - 0.5 \cdot t_{sip} \qquad d_p = 6.25\,in$$

$W_{sip} = 96\,in$ effective width of compression flange

$$\beta_1 := \text{if}\left[f'_{cs} \le 4 \cdot ksi, 0.85, 0.85 - 0.05 \cdot \left(\frac{f'_{cs} - 4.0 \cdot ksi}{1.0 \cdot ksi}\right)\right] \qquad \beta_1 := \begin{cases} \beta_1 & \text{if } \beta_1 \ge 0.65 \\ 0.65 & \text{otherwise} \end{cases}$$

$\beta_1 = 0.85$ (§5.7.2.2)

Assume rectangular section,

$$c := \frac{A_{ps} \cdot f_{pu}}{0.85 \cdot f'_{cs} \cdot \beta_1 \cdot W_{sip} + k \cdot A_{ps} \cdot \frac{f_{pu}}{d_p}} \qquad c = 1.47\,in$$

Stress in prestressing steel at nominal flexural resistance, f_{ps} (§5.7.3.1.1),

$$f_{ps} := f_{pu} \cdot \left(1 - k \cdot \frac{c}{d_p}\right) \qquad f_{ps} = 252.24\,ksi$$

Check stress in prestressing steel according to available development length, l_d

Available development length at midspan of the SIP panel,

$$l_d := 0.5 \cdot L_{sip} \qquad l_d = 0.966\,m$$

rearranging LRFD eq. 5.11.4.1-1

$$f_{psld} := \frac{l_d}{1.6 \cdot d_b} \cdot ksi + \frac{2}{3} \cdot f_{pe} \qquad f_{psld} = 177.404\,ksi$$ (may be too conservative)

$$f_{ps} := \min\left(\left(f_{ps} \;\; f_{psld}\right)\right) \qquad f_{ps} = 177.404\,ksi$$

Flexural Resistance (§5.7.3.2.2 & 5.7.3.2.2),

$a := \beta_1 \cdot c$ $\quad a = 1.25\,in$ $\quad A_{ps} = 1.615\,in^2$ per panel

$$M_n := A_{ps} \cdot f_{ps} \cdot \left(d_p - \frac{a}{2}\right)$$

$$M_n = 134.3\,kip \cdot ft$$

$M_r := \phi_p \cdot M_n$ $\quad M_r = 134.3\,kip \cdot ft$ per panel

$$M_r := \frac{M_r}{W_{sip}} \qquad M_r = 16.79\,\frac{kip\cdot ft}{ft} \qquad \text{per ft}$$

$M_u \le M_r = 1$ **OK** where $M_u = 9.874\,\frac{kip\cdot ft}{ft}$

Limits of Reinforcement

Maximum Reinforcement (§5.7.3.3.1)

$$d_e := \frac{A_{ps}\cdot f_{ps}\cdot d_p}{A_{ps}\cdot f_{ps}} \qquad d_e = 6.25\,in$$

The maximum amount of prestressed and non-prestressed reinforcement shall be such that

$\frac{c}{d_e} \le 0.42 = 1$ OK, where $\frac{c}{d_e} = 0.23$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

Minimum Reinforcement (§5.7.3.3.2)

AASHTO 9.18.2

Compressive stress in concrete due to effective prestress force (after all losses) at midspan

$$f_{peA} := \frac{P_e\cdot ft}{A_{sip}} \qquad f_{peA} = 0.82\,ksi \qquad \text{(compression)}$$

Non-composite dead load moment at section, M_{dnc},

$$M_{dnc} := M_{cip} + M_{sip} \qquad M_{dnc} = 0.569\,\frac{kip\cdot ft}{ft}$$

$f_r = 0.537\,ksi$ use SIP panel

$$M_{cr} := \left(f_r + f_{peA}\right)\cdot\frac{S_b}{ft} - M_{dnc}\cdot\left(\frac{S_b}{S_{bp}} - 1\right) \qquad 1.2\cdot M_{cr} = 14.114\,\frac{kip\cdot ft}{ft}$$

$M_r \ge 1.2\cdot M_{cr} = 1$ **OK** where $M_r = 16.79\,\frac{kip\cdot ft}{ft}$

Negative Moment Section Over Interior Beams

Deck shall be subdivided into strips perpendicular to the supporting components (§4.6.2.1.1).
Continuous beam with span length as center to center of supporting elements (§4.6.2.1.6).
Wheel load may be modeled as concentrated load or load based on tire contact area.
Strips should be analyzed by classical beam theory.

Spacing in secondary direction (spacing between diaphragms):

$$L_d := \frac{L}{1.0} \qquad L_d = 27.149\,m$$

Spacing in primary direction (spacing between girders):

$S = 2.057\,m$

Since $\frac{L_d}{S} \geq 1.50 = 1$, where $\frac{L_d}{S} = 13.2$ (§4.6.2.1.5)

therefore, all the wheel load shall be applied to primary strip. Otherwise, the wheels shall be distributed between intersecting strips based on the stiffness ratio of the strip to sum of the strip stiffnesses of intersecting strips.

Critical Section

The design section for negative moments and shear forces may be taken as follows:

Prestressed girder - shall be at 1/3 of flange width < 15 in.

Steel girder - 1/4 of flange width from the centerline of support.

Concrete box beams - at the face of the web.

top flange width $b_f := 15.06 \cdot in$

Design critical section for negative moment and shear shall be at d_c, (§4.6.2.1.6)

$d_c := \min\left(\left(\frac{1}{3} \cdot b_f \quad 15 \cdot in\right)\right)$ $d_c = 5\,in$ from CL of girder (may be too conservative, see training notes)

Maximum factored moments **per unit width** based on Table A4-1: for $S = 2.057\,m$

(include multiple presence factors and the dynamic load allowance)

applicability $\text{if}\left[\min((0.625 \cdot S \quad 6 \cdot ft)) \geq overhang - cw, \text{"OK"}, \text{"NG"}\right] = \text{"OK"}$

$\text{if}\left(N_b \geq 3, \text{"OK"}, \text{"NG"}\right) = \text{"OK"}$

$M_{LLn} := 4.00 \cdot \frac{kip \cdot ft}{ft}$ (max. -M at d_c from CL of girder)

Dead load moment (STRUDL s-dl output)

$M_{DCn} := 0.18 \cdot \frac{kip \cdot ft}{ft}$ (dead load from deck overhang and sidl only, max. -M at d_c at interior girder, conservative) $\frac{d_c}{S} = 0.062$

$M_{wsn} := 0 \cdot \frac{kip \cdot ft}{ft}$

Service negative moment

$M_{sn} := M_{DCn} + M_{wsn} + M_{LLn}$ $M_{sn} = 4.18 \frac{kip \cdot ft}{ft}$

Factored negative moment

$$M_{un} := \eta \cdot \left(\gamma_p \cdot M_{DCn} + \gamma_{DW} \cdot M_{wsn} + \gamma_L \cdot M_{LLn}\right) \qquad M_{un} = 7.23 \frac{kip \cdot ft}{ft}$$

Design of Section

Normal flexural resistance of a rectangular section may be determined by using equations for a flanged section in which case b_w shall be taken as b (§5.7.3.2.3).

$$\beta_1 := \text{if}\left[f'_{cs} \le 4 \cdot ksi, 0.85, 0.85 - 0.05 \cdot \left(\frac{f'_{cs} - 4.0 \cdot ksi}{1.0 \cdot ksi}\right)\right] \qquad \beta_1 := \begin{vmatrix} \beta_1 & \text{if } \beta_1 \ge 0.65 \\ 0.65 & \text{otherwise} \end{vmatrix}$$

$\beta_1 = 0.85$ (§5.7.2.2) conservatively use CIP slab concrete strength

assume bar # $bar_n := 5$

$$dia(bar) := \begin{vmatrix} 0.5 \cdot in & \text{if } bar = 4 \\ 0.625 \cdot in & \text{if } bar = 5 \\ 0.75 \cdot in & \text{if } bar = 6 \\ 0.875 \cdot in & \text{if } bar = 7 \end{vmatrix}$$

$$d_n := t_{s2} - 2.5 \cdot in - \frac{dia(bar_n)}{2} \qquad d_n = 5.688\, in$$

$$A_s := \frac{0.85 \cdot f'_{cs} \cdot ft}{f_y} \cdot \left(d_n - \sqrt{d_n^{\,2} - \frac{2 \cdot M_{un} \cdot ft}{0.85 \cdot \phi_f \cdot f'_{cs} \cdot ft}}\right) \qquad A_s = 0.29\, in^2 \quad \text{per ft}$$

use (top-transverse) bar # $bar_n = 5$ $s_n := 9 \cdot in$

$$A_b(bar) := \begin{vmatrix} 0.20 \cdot in^2 & \text{if } bar = 4 \\ 0.31 \cdot in^2 & \text{if } bar = 5 \\ 0.44 \cdot in^2 & \text{if } bar = 6 \\ 0.60 \cdot in^2 & \text{if } bar = 7 \end{vmatrix}$$

$$A_{sn} := A_b(bar_n) \cdot \frac{1 \cdot ft}{s_n} \qquad A_{sn} = 0.41\, in^2 \quad \text{per ft}$$

Maximum Reinforcement (§5.7.3.3.1)

The max. amount of prestressed and non-prestressed reinforcement shall be such that

where $d_e := d_n$

$$c := \frac{A_{sn} \cdot f_y}{0.85 \cdot \beta_1 \cdot f'_{cs} \cdot 1 \cdot ft} \qquad c = 0.72\,in$$

$$\text{if}\left(\frac{c}{d_e} \le 0.42, \text{"OK"}, \text{"NG"}\right) = \text{"OK"} \qquad \frac{c}{d_e} = 0.126$$

The section is not over-reinforced. Over-reinforced reinforced concrete sections shall not be permitted.

Minimum Reinforcement (§5.7.3.3.2)

$$f_{rs} := 0.24 \cdot \sqrt{\frac{f'_{cs}}{ksi}} \cdot ksi \qquad f_{rs} = 0.48\,ksi$$ use SIP panel concrete strength

$$n := \frac{E_s}{E_{cs}} \qquad n = 6.866 \qquad n := \max[\,[\,\text{ceil}((n - 0.495)) \;\; 6\,]\,]$$

$n = 7$ set $n = 7$ (round to nearest integer, §5.7.1, not less than 6)

$$(n - 1)A_{sn} = 2.48\,in^2$$

$$A_{gc} := t_{s2} \cdot ft \qquad A_{gc} = 102\,in^2$$

$d_s := 2.5in + 0.625 \cdot in + 0.5 \cdot 0.75 \cdot in$ c.g. of reinforcement to top of slab $d_s = 3.5\,in$

$$Y_{ts} := \frac{A_{gc} \cdot 0.5 \cdot t_{s2} + (n - 1) \cdot A_{sn} \cdot d_s}{A_{gc} + (n - 1) \cdot A_{sn}} \qquad Y_{ts} = 4.232\,in$$

$$I_{cg} := \frac{ft \cdot (t_{s2})^3}{12} + A_{gc} \cdot (0.5 \cdot t_{s2} - Y_{ts})^2 + (n - 1)A_{sn} \cdot (Y_{ts} - d_s)^2 \qquad I_{cg} = 615.487\,in^4$$

$$M_{cr} := \frac{f_{rs} \cdot I_{cg}}{Y_{ts}} \qquad M_{cr} = 5.817\,kip \cdot ft \qquad 1.2 \cdot M_{cr} = 6.981\,kip \cdot ft$$

$$\text{if}(M_{un} \cdot ft \ge 1.2 \cdot M_{cr}, \text{"OK"}, \text{"NG"}) = \text{"OK"}$$

Crack Control (§5.7.3.4)

$$M_{sn} = 4.18 \frac{kip \cdot ft}{ft}$$

$$n := \frac{E_s}{E_{cs}} \qquad n = 6.866 \qquad n := \text{ceil}((n - 0.495))$$ use slab concrete strength

set $n = 7$ (round to nearest integer, §5.7.1)

$$\rho := \frac{A_{sn}}{ft \cdot d_n} \qquad \rho = 6.056 \times 10^{-3}$$

$$k(\rho) := \sqrt{(\rho \cdot n)^2 + 2 \cdot \rho \cdot n} - \rho \cdot n \qquad k(\rho) = 0.252$$

$$j(\rho) := 1 - \frac{k(\rho)}{3} \qquad j(\rho) = 0.916$$

$$f_{sa} := \frac{M_{sn} \cdot ft}{A_{sn} \cdot j(\rho) \cdot d_n} \qquad f_{sa} = 23.293\,ksi$$

$Z := 130 \cdot \frac{kip}{in}$ crack width parameter in severe exposure condition

$d_c := 2 \cdot in + 0.5 \cdot dia(bar_n)$ $\quad d_c = 2.313\,in$ max.

$$A := 2 \cdot d_c \cdot s_n \qquad A = 41.625\,in^2$$

$$\text{if}\left[\min\left[\left[\frac{Z}{(d_c \cdot A)^{\frac{1}{3}}}\right]\right] \ge f_{sa}, \text{"OK"}, \text{"NG"}\right] = \text{"OK"} \qquad \text{where} \quad \min\left[\left[\frac{Z}{(d_c \cdot A)^{\frac{1}{3}}}\right]\right] = 28.366\,ksi$$

$$\qquad\qquad\qquad\qquad 0.6 \cdot f_y \qquad\qquad\qquad\qquad\qquad\qquad\qquad 0.6 \cdot f_y$$

say OK

Shrinkage and Temperature Reinforcement (§5.10.8.2)

For components less than 48 in. thick,

where $\quad A_g := t_{s2} \cdot 1 \cdot ft$

$$A_{tem} := 0.11 \cdot \frac{A_g \cdot ksi}{f_y} \qquad A_{tem} = 0.187\,in^2 \quad \text{per ft}$$

The spacing of this reinforcement shall not exceed $3 \cdot t_{s1} = 24\,in$ or 18 in.

top longitudinal - $bar := 4$ $\quad s := 12 \cdot in$ $\quad A_s := A_b(bar) \cdot \frac{1 \cdot ft}{s}$ $\quad A_s = 0.2\,in^2$ per ft **OK**

Distribution of Reinforcement (§9.7.3.2)

The effective span length S_{eff} shall be taken as (§9.7.2.3):

web thickness $\quad b_w := 7 \cdot in$

top flange width $\quad b_f = 15.06\,in$

$$S_{eff} := S - b_f + \frac{b_f - b_w}{2} \qquad S_{eff} = 5.831\,ft$$

For primary reinforcement perpendicular to traffic:

$$percent := \min\left(\left(\frac{220}{\sqrt{\frac{S_{eff}}{ft}}} \quad 67\right)\right) \qquad percent = 67$$

Bottom longitudinal reinforcement (convert to equivalent mild reinforcement area):

$$A_s := \frac{percent}{100} \cdot \frac{A_{ps}}{W_{sip}} \cdot \frac{f_{py}}{f_y} \qquad A_s = 0.55\frac{in^2}{ft} \qquad \text{per ft}$$

use bar # $bar := 5$ $s := 6.0 \cdot in$ $A_s := A_b(bar) \cdot \frac{1 \cdot ft}{s}$ $A_s = 0.62\,in^2$ per ft **OK**

Maximum bar spacing (§5.10.3.2)

Unless otherwise specified, the spacing of the primary reinforcement in walls and slabs shall not execeed 1.5 times the thickness of the member or 18 in.. The maximum spacing of temperature shrinkage reinforcement shall be as specified in §5.10.8.

$1.5 \cdot t_{s1} = 12\,in$ **OK**

Protective Coating (§5.12.4)

Epoxy coated reinforcement shall be used for slab top layer reinforcements except when the slab is overlayed with asphalt.

Ribbed Girder Design Example
5' Wide Trideck, 65' Span Length

All dimensions are in kips and inches except bending moments which are measured in kip-feet

Material Properties

<u>Concrete</u>

$f_c := 8.0$

$f_{ci} := 6.5$

$E_c := 5696$

$E_{ci} := 5134$

$f_r := 0.68$

$\rho := 160$

$\mu := 0.2$

<u>Prestressing Steel</u>

$f_{pu} := 270$

$f_{py} := 243$

$E_p := 28500$

<u>Reinforcing Steel</u>

$f_y := 60$

$E_s := 29000$

<u>Composite Action</u>

$n := \dfrac{E_p}{E_c}$

$n_i := \dfrac{E_p}{E_{ci}}$

Geometric Properties of Tribeam

$A_g := 746$ $\quad y_b := 17.16$

$I_g := 46406$ $\quad h := 27$

$S_b := \dfrac{I_g}{y_b}$ $\quad b := 60$ (5' wide section)

$S_t := \dfrac{I_g}{(h - y_b)}$ $\quad L := 65 \cdot 12$ (65' span)

Permanent Loads

$w_{dl} := 0.829$

$w_{sdl} := 0.175$

} kips/ft

(assumes 3' overlay)

Design Load Effects

(calculated at midspan)

$M_{dl} := \dfrac{w_{dl}}{8} \cdot \left(\dfrac{L}{12}\right)^2$ $\quad M_{dl} = 437.82$

$M_{sdl} := \dfrac{w_{sdl}}{8} \cdot \left(\dfrac{L}{12}\right)^2$ $\quad M_{sdl} = 92.42$

$M_{ll} := 2 \cdot .227 \cdot 1525$ $\quad M_{ll} = 692.35$ (LL DF for two loaded lanes)

$M_{serviceI} := M_{dl} + M_{sdl} + M_{ll}$ $\quad M_{serviceI} = 1222.59$

$M_{serviceIII} := M_{dl} + M_{sdl} + 0.8 \cdot M_{ll}$ $\qquad M_{serviceIII} = 1084.12$

Prestressing Geometry

$A_{str} := 0.153$ $\qquad y_{bps} := 5.0$ $\qquad$ (y_{bps} is "F")

$d_{strand} := 0.5$ $\qquad e := y_b - y_{bps}$

$N_{strand} := 33$ $\qquad A_{ps} := N_{strand} \cdot A_{str}$ $\qquad A_{ps} = 5.05$

Transformed Section Properties (between harp pts.)

$A_{net} := A_g - A_{ps}$ $\qquad A_{net} = 740.95$

$$y_{bt} := \frac{\left[y_b A_{net} + y_{bps} \cdot (n-1) \cdot A_{ps}\right]}{A_{net} + (n-1) \cdot A_{ps}} \qquad y_{bt} = 16.84$$

$$I_t := I_g + A_g \cdot \left(y_b - y_{bt}\right)^2 + (n-1) \cdot A_{ps} \cdot \left(y_{bt} - y_{bps}\right)^2 \qquad I_t = 49316.07$$

$$S_{bt} := \frac{I_t}{y_{bt}} \qquad S_{bt} = 2929.02$$

$$S_{tt} := \frac{I_t}{h - y_{bt}} \qquad S_{tt} = 4852.55$$

$e_t := y_{bt} - y_{bps}$ $\qquad e_t = 11.84$

Prestress Forces

$f_{pi} := 0.75 \cdot f_{pu}$

$P_{jack} := 0.75 \cdot f_{pu} \cdot A_{ps}$ $\qquad P_{jack} = 1022.42$ (strand jacking force)

$P_{si} := 0.69 \cdot f_{pu} \cdot A_{ps}$ $\qquad P_{si} = 940.63$ (initial p/s force)

Elastic Shortening Losses

$$f_{cgp} := \left(\frac{P_{si}}{A_{net}}\right) + \left(P_{si} \cdot \frac{e^2}{I_g}\right) - \left(12 M_{dl} \cdot \frac{e_t}{I_g}\right) \qquad f_{cgp} = 2.93$$

$$\Delta f_{pinstant} := \left(\frac{E_p}{E_{ci}}\right) \cdot f_{cgp} \qquad \Delta f_{pinstant} = 16.25$$

Time Dependent Losses

$$\Delta f_{td} := 37 \cdot \left[1 - 0.15 \cdot \frac{\left(f_c - 6\right)}{6}\right] \qquad \Delta f_{td} = 35.15$$

$\Delta f_{total} := \Delta f_{pinstant} + \Delta f_{td}$ $\qquad \Delta f_{total} = 51.40$

$P_{se} := \left(f_{pi} - \Delta f_{total}\right) \cdot A_{ps}$ $\qquad \boxed{P_{se} = 762.92}$ (p/s force after losses)

Service I and Service III Limit States

Bottom Fiber Stress at Midspan: (compression is negative)

$$f_b := P_{se}\cdot\left(\frac{e}{S_b}+\frac{1}{A_{net}}\right)-\left(\frac{12M_{serviceIII}}{S_{bt}}\right)$$

$\boxed{f_b = 0.02}$ 0 Allowable

(BDM 6.2.3-2)

Top Fiber Stress at Midspan

Allowable

$$f_{tI} := P_{se}\cdot\left(\frac{1}{A_{net}}-\frac{e}{S_t}\right)+12\cdot\frac{M_{dl}+M_{sdl}}{S_{tt}}$$

$\boxed{f_{tI} = 0.37}$ $0.45\cdot f_c = 3.60$

$$f_{tII} := P_{se}\cdot\left(\frac{1}{A_{net}}-\frac{e}{S_t}\right)+12\cdot\left(\frac{.5M_{dl}+M_{ll}}{S_{tt}}\right)$$

$\boxed{f_{tII} = 1.32}$ $0.4\cdot f_c = 3.20$

$$f_{tIII} := P_{se}\cdot\left(\frac{1}{A_{net}}-\frac{e}{S_t}\right)+12\cdot\frac{M_{serviceI}}{S_{tt}}$$

$\boxed{f_{tIII} = 2.09}$ $0.6\cdot f_c = 4.80$

(BDM 6.2.3-2)

Strength Limit State

Strength I Load Effect at Midspan

$$M_u := 1.25\cdot M_{dl}+1.5\cdot M_{sdl}+1.75\cdot M_{ll}$$

$\boxed{M_u = 1897.51}$

(LRFD 3.4.1-1)

Bonded Steel Stress

$k := 0.28$ low relaxation steel

$$c := \frac{\left(A_{ps}\cdot f_{pu}\right)}{\left[0.85\cdot 0.85\cdot f_c\cdot b+k\cdot A_{ps}\cdot\frac{f_{pu}}{\left(h-y_{bps}\right)}\right]}$$

$c = 3.74$

$a := 0.85\cdot c$

$$f_{ps} := f_{pu}\cdot\left[1-k\cdot\frac{c}{\left(h-y_{bps}\right)}\right]$$

$f_{ps} = 257.14$

Flexural Capacity at Midspan

$\phi := 1.0$

$$\phi M_n := \phi\cdot A_{ps}\cdot f_{ps}\cdot\left(h-y_{bps}-\frac{a}{2}\right)\cdot\frac{1}{12}$$

$\boxed{\phi M_n = 2208.04}$

$\phi M_n > M_u$ **OK**

Service I Limit State, Prestressing Steel

$$\Delta f_{ps} := n \cdot \left(\frac{e}{y_{bt}}\right) \cdot \left[12 \cdot \frac{(M_{ll} + M_{sdl})}{S_{bt}}\right] \qquad \Delta f_{ps} = 11.62$$

$$f_{psservice} := f_{pi} - \Delta f_{total} + \Delta f_{ps} \qquad \boxed{f_{psservice} = 162.72}$$

$0.8 \cdot f_{py} = 194.40$ maximum **(LRFD 5.9.3)** **OK**

Reinforcement Limits

Maximum RF

$$\frac{c}{(h - y_{bps})} = 0.17$$

0.42 maxmimum **(LRFD 5.7.3.3.1-1)** **OK**

Mimumum RF

$$f_{cpe} := P_{se} \cdot \left(\frac{1}{A_{net}} + \frac{e}{S_b}\right) \qquad f_{cpe} = 4.46$$

$$M_{cr} := \frac{S_{bt} \cdot (f_{cpe} - f_r)}{12} \qquad 1.2M_{cr} = 1107.22$$

$$1.33 \cdot M_u = 2523.69$$

$\phi M_n = 2208.04$ Greater than the lesser of 1.2 M_{cr} and 1.33 M_u **(LRFD 5.7.3.3.2)** **OK**

Transformed Section at Endblock (at transfer, $E=E_{ci}$)

$y_{bsend} := 12$ (y_b of steel at end block)

$$e_{end} := y_b - y_{bsend} \qquad e_{end} = 5.16$$

$$y_{btend} := \frac{[y_b A_g + (n_i - 1) \cdot y_{bsend} \cdot A_{ps}]}{(n_i - 1) \cdot A_{ps} + A_g} \qquad y_{btend} = 17.01$$

$$I_{tend} := I_g + A_g \cdot (y_b - y_{btend})^2 + (n_i - 1) \cdot A_{ps} \cdot (y_{btend} - y_{bsend})^2 \qquad I_{tend} = 46999.55$$

$$S_{btend} := \frac{I_{tend}}{y_{btend}} \qquad S_{btend} = 2763.74$$

$$S_{ttend} := \frac{I_{tend}}{h - y_{btend}} \qquad S_{ttend} = 4702.69$$

$e_{tend} := y_{btend} - y_{bsend}$ $\qquad e_{tend} = 5.01$

Concrete Stresses at Transfer

$P_i := \left(f_{pi} - \Delta f_{pinstant}\right) \cdot A_{ps}$ $\qquad P_i = 940.40$

$l_t := 60 \cdot d_{strand}$

$y_{bpslt} := y_{bps} + \left(y_{bsend} - y_{bps}\right) \cdot \left[\frac{\left(L - l_t\right)}{L}\right]$ $\qquad y_{bpslt} = 11.73$

$e_{lt} := y_{btend} - y_{bpslt}$ $\qquad e_{lt} = 5.28$

P/S Load Effects at Transfer

$M_{harp} := w_{dl} \cdot \frac{3 \cdot \left(\frac{L}{12}\right)^2}{25}$ $\qquad M_{harp} = 420.30$

$M_{lt} := \frac{w_{dl}}{144} \cdot l_t \cdot \left(L - l_t\right)$ $\qquad M_{lt} = 129.53$

Allowable Stresses:

Tension < -0.200 $\qquad$ Compression $\quad 0.6 \cdot f_{ci} = 3.90$ $\qquad$ **(BDM 6.2.3-2)**

Top Fiber at Transfer Length from Endblock

$f_{tlt} := P_i \cdot \left(\frac{1}{A_{net}} - \frac{e_{lt}}{S_t}\right) + 12 \cdot \frac{M_{lt}}{S_{ttend}}$ $\qquad \boxed{f_{tlt} = 0.183}$ $\qquad$ **OK**

Bottom Fiber at Transfer Length

$f_{blt} := P_i \cdot \left(\frac{1}{A_{net}} + \frac{e_{lt}}{S_b}\right) - 12 \cdot \frac{M_{lt}}{S_{btend}}$ $\qquad \boxed{f_{blt} = 2.54}$ $\qquad$ **OK**

Top Fiber at Harp Point

$f_{tharp} := P_i \cdot \left(\frac{1}{A_{net}} - \frac{e}{S_t}\right) + \frac{12 \cdot M_{harp}}{S_{tt}}$ $\qquad \boxed{f_{tharp} = -0.12}$ $\qquad$ **OK**

Bottom Fiber at Harp Point

$f_{bharp} := P_i \cdot \left(\frac{1}{A_{net}} + \frac{e}{S_b}\right) - 12 \cdot \frac{M_{harp}}{S_{bt}}$ $\qquad \boxed{f_{bharp} = 3.78}$ $\qquad$ **OK**

Camber and Deflection

At Release:

Prestress Effect:

$$a := 0.3 \cdot L$$

$$e_h := y_b - y_{bps} - y_{bsend}$$

$$\Delta_{ps} := \frac{P_{si}}{E_{ci} \cdot I_t} \cdot \left(\frac{e \cdot L^2}{8} - \frac{e_h \cdot a^2}{6} \right) \qquad \Delta_{ps} = 3.43$$

Dead Load Effect:

$$\Delta_{dl} := -\frac{5 \cdot \frac{w_{dl}}{12} \cdot L^4}{384 \cdot E_{ci} \cdot I_t} \qquad \Delta_{dl} = -1.32$$

Superimposed Loads

$$\Delta_{SDL} := \frac{-5 \cdot \frac{w_{sdl}}{12} \cdot L^4}{384 \cdot E_{ci} \cdot I_t} \qquad \Delta_{SDL} = -0.28$$

$\Delta 1$

$$\Delta_1 := \Delta_{ps} + \Delta_{dl} \qquad \Delta_1 = 2.12$$

$\Delta 2000$

$$\Delta_{2000} := 2.70 \cdot \Delta_{dl} + 2.45 \cdot \Delta_{ps} \qquad \Delta_{2000} = 4.85$$

ΔSDL

$$\Delta_{SDL} := 3.00 \cdot \Delta_{SDL} \qquad \Delta_{SDL} = -0.83$$

Excess Camber: $\boxed{\Delta_{2000} + \Delta_{SDL} = 4.02}$ Multipliers from **BDM 6.1.8**

Prestressed Voided Slab Design
AASHTO LRFD Specifications

General Input							Specification Reference
Girder type:		26 Inch Prestressed Precast Voided Slab					Prelim.Plan, Sh 1
Span Length:	L =	45.00	ft	C.L. to C.L. Bearing			BDM 6.3.1A
Bridge Width:		44.00	ft	Deck Width			Prelim.Plan, Sh 1
Number of Lanes		3					"
Skew Angle:	θ_{skew} =	0.00	degrees				"
Girder Width:	b =	4.00	ft =	48.00	in		BDM 6.6-A3-1
Girder Depth:	h or d =	26	in =	2.17	ft	OK	"
Number of Voids	N_v =	2					"
Void Diameter:	dia_v =	15.7	in				"
Number of girders:	N_b =	11		Assumed ACP added =			Prelim.Plan Sh 2
Weight of barrier:	w_{TB} =	0.50	k/ft		0.20	in.	BDM 8.3-A10
Weight of Overlay (3" Asphalt)		0.040	k/ft^2	$ACP_{min.}$ =	3.00	in.	
Strength of Concrete				$ACP_{applied}$ =	3.20	in.	
Final	f_c' =	7.0	ksi	Transfer f_{ci}' =	6.0	ksi	00 BDM 6.6-A3-2

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Prestressing Input					Reference
Strand diam., d_b = 0.50 in	Area =	0.153	in^2		BDM 6.6-A3-1
Number of Bonded Strands ~2 in from Bottom		14			See Strand Layout Worksheet
Number of Bonded Strands ~4 in from Bottom		0	Eccentricity (E)		
Number of Bonded Strands ~6 in from Bottom		0	E =	7.18 in	
Number of Debonded Bottom Strands		4	OK	OK	
Total Number of Bottom Strands		18	≤	42	OK
Total Number of Top Strands		4	≤	6	OK

Output				
HS20-44 Force Effect:		**Jacking Force, P_j =** 691 kips		See Section 9
Live Load Force Effect:	Moment =	878.49	ft-kips per lane	See Section 7
	Reaction =	90.30	kips per lane	

Service Limit State

		Top of Girder		Bottom of Girder		**Check**
Concrete Stresses at Transfe	Load Case	Calculated	Allowable	Calculated	Allowable	
At d_v	DL+P/S	-0.014	0.200	-1.249	-3.600	OK
Concrete Stresses at Service		**Limit State I**		**Limit State III**		
At mid-span	DL+LL+P/S	-1.407	-4.200	-0.030	0.000	OK
		-	-	-0.030	0.251	OK
	DL+P/S	-0.628	-3.150	-	-	OK
	LL+1/2DL+1/2P/S	-1.093	-2.800	-	-	OK

Strength Limit State

Moment at Mid-span, ft-kips	Mu =	978	ϕ Mn =	1351.2	OK

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Final stress at bottom of girder

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Stresses at transfer

Determination of prestressing losses at transfer

Total loss at transfer

Concrete stress immediately after transfer at dv f

11 Strength Limit State

Resistance factor

Flexural forces — NG Mu No Check

Flexural resistance — OK for rectangular section

Nominal flexural resistance

Limit for reinforcement

Maximum reinforcement — OK Not overreinforced

Minimum reinforcement

Development of prestressing strand — OK developed

12 Shear Design

Design procedure

Effective Web Width, bv, and Effective Shear Depth, dv

Component of Prestressing Force in Direction of Shear Force, Vp

Shear Stress Ratio

Factored shear force

fpo

Factored moment

Longitudinal Strain (Flexural Tension), ε_x

Determination of β and θ

Shear strength

Required shear strength

Maximum spacing of shear reinforcement

Minimum shear reinforcement — OK for Min. Transverse Reinf.

Longitudinal reinforcement — OK for Longitudinal Reinforcement

13 Deflection and Camber

Prestressed Voided Slab Design
AASHTO LRFD Specifications

Specification Reference

1 Structure: **Project XLXXXX, Name Br #XX/XX**

Single Span Bridge

				Specification Reference
Span Length:	45.00	ft	C.L. / C.L. Bearing / Bkps	Prelim Plan, Sh 1
Girder Length:	45.83	ft		
Bridge Width:	44.00	ft	Deck between curbs &/or barriers	"
Girder Width:	4.00	ft		BDM6.6-A3-1
Number of girders:	11			Prelim Plan, Sh 1

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2 Live load — LRFD 3.6.1

HL-93

Vehicular live load designated as " HL-93 " shall consist of a combination of : — LRFD 3.6.1.2.1

Design truck or design tandem, plus

Design lane load

Design truck is equivalent to AASHTO HS20-44 truck. — LRFD 3.6.1.2.2

The design lane shall consist of a 0.64 klf, uniformly distributed in the longitudinal direction. Design lane load shall be assumed to be uniformly distributed over 10 ft width in the transverse direction. — LRFD 3.6.1.2.4

Design tandem shall consist of a pair of 25.0 kip axles spaced at 4'-0" apart — LRFD 3.6.1.2.3

Number of design lanes: — LRFD 3.6.1.1.1

Integer part of : Width / (12 ft lane) = 3 Lanes

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3 Material Properties

Concrete

LRFD Specifications allows a concrete compressive strength with a range of 2.4 to 10.0 ksi at 28 days. — LRFD 5.4.2.1

Compressive strength for prestressed concrete shall not be less than 4.0 ksi.

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Precast prestressed girder — 2000 BDM, 6.6-A3-2

Compressive strength at 28 days,		f_c' =	7.0	ksi
Compressive strength at transfer,		f_{ci}' =	6.0	ksi
Unit weight,	(for computing, E_c)	w_c =	0.160	kcf

(for DL calculation)	$w_c =$	0.160	kcf	BDM 4.1.1 & 4/18/00 Bridge Design Memo
*Modulus of Elasticity,	$E_c = 33000 w_c^{1.5}\sqrt{f_c'} =$	5587.8	ksi	LRFD Eqn 5.4.2.4-1

*LRFD states this equation is for concretes with unit weights between 0.090 and 0.155 kcf. WSDOT uses 0.160kcf. Assume this is still ok. $0.90 \le w_c \le 0.155$

Modulus of Rupture,	$f_r = 0.24\sqrt{f_c'} =$	0.635	ksi	LRFD 5.4.2.6
Poisson's ratio =	$\mu =$	0.2		LRFD 5.4.2.5

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Reinforcing steel — LRFD 5.4.3

AASHTO M-31 with yield strength of,	$f_y =$	60.00	ksi	LRFD 5.4.3.1 & BDM 5.1.2A
Modulus of Elasticity,	$E_s =$	29000	ksi	LRFD 5.4.3.2

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Prestressing steel — LRFD 5.4.4

AASHTO M-203, Uncoated 0.5 in. or 0.6 in. diameter, Low-relaxation — LRFD 5.4.4.1

Ultimate strength,	$f_{pu} =$	270	ksi.	LRFD Table 5.4.4.1-1
Yield strength,	$f_{py} = 0.9\, f_{pu} =$	243	ksi.	"
Modulus of elasticity,	$E_p =$	28500	ksi.	LRFD 5.4.4.2

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4 Allowable Concrete Stresses at Service Lmit State — LRFD 5.9.4

Tensile stress limit — LRFD 5.9.4.2.2

For service loads which involve traffic loading, tensile stress in members with bonded prestressing strands shall be investigated using Service - III load combination.

Tension in precompressed tensile zone assuming uncracked section:

$$f_t = 0.19\sqrt{f_c'}$$ Areas with bonded reinforcement — LRFD Table 5.9.4.2.2-1

$$f_t = 0.0948\sqrt{f_c'}$$ Components subjected to severe corrosive conditions

Current WSDOT design practice does not allow any tension at the bottom of prestressed girders, therefore:

$f_t =$ 0.00 ksi — B.K.

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Compressive stress limits after all losses LRFD 5.9.4.2

Compression shall be investigated using Service - I load combination:

$f_c = 0.45\ f'_c$ Due to permanent loads

$f_c = 0.60\ f'_c$ Due to permanent and transient loads

$f_c = 0.40\ f'_c$ Due to transient loads and one-half of permanent loads

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Stress limits for prestressing strands

Ultimate tensile strength,	f_{pu} =	270.00	ksi
Yield strength,	$f_{py} = 0.9\ f_{pu}$ =	243.00	ksi
Immediately <u>prior</u> to Transfer	$f_{pbt} = 0.75\ f_{pu}$ =	202.50	ksi

LRFD Table 5.4.4.1-1

Effective stress limit at Service Limit State after all losses:

$f_{pe} = 0.8\ f_{py}$ = 194.40 ksi

LRFD Table 5.9.3-1

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5 Computation of Section Properties

Table 5-1: Section Area

Gross Section	Width	48.00	in.	BDM 6.6-A3-1
	Depth	26.00	in	"
Void	Number	2	in	"
	Diameter	15.70	in	"
Net Web = Width - Voids = b_v =		24.45	in	See Section 12

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For both pretensioned or posttensioned members after bonding of tendons, section properties may be based on either the gross or transformed section. The following section properties are based on gross section (of the concrete). LRFD 5.9.1.4

Table 5-2: Moment of Inertia, I

	Area in^2	Moment of Inertia in^4		
Gross	1248.00	70304.00	$I_{rectangle} = 1/12\ bh^3$	AISC p 7-17
Void	387.19	5964.84	$I_{circle} = 1/4\ \pi\ r^4$	AISC p 7-20
Net	860.81	64339.16	(Gross Concrete Section)	

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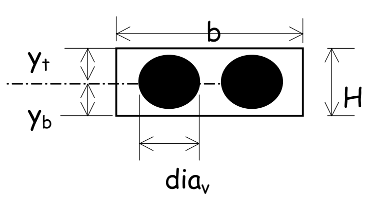
CG=NA



Location of the Neutral Axis: $$y_b = \frac{\sum yA}{\sum A} = 13.00 \text{ in}$$

$$y_t = H - y_b = 13.00 \text{ in}$$

Section Modulus:	(Bottom)	$S_b = \frac{I}{y_b} =$	4949.2	in^3	BDM 6.4-A3-1
	(Top)	$S_t = \frac{I}{y_t} =$	4949.2	in^3	"

6 Limit State

LRFD 1.3.1

Each component and connection shall satisfy the following equation for each limit state :

$$\sum \eta_i \gamma_i Q_i \le \phi R_n = R_r$$

LRFD Eqn. 1.3.2.1-1

Where:

Load Modifier for Ductility, Redundancy, & Operational Importance

η_i = $\eta_i \eta_R \eta_I \ge 0.95$ for loads which a max. value of γ_i is appropriate — LRFD Eqn. 1.3.2.1-2

= $\dfrac{1}{\eta_D \eta_R \eta_I} \le 1.00$ for loads which a min. value of γ_i is appropriate — LRFD Eqn. 1.3.2.1-3

η_D =	Ductility factor	LRFD 1.3.3
η_R =	Redundancy factor	LRFD 1.3.4
η_I =	Operational Importance factor	LRFD 1.3.5
η_i = 1.00	WSDOT Bridge office practice: for any ordinary structure	B.K.

Therefore the Limit State Equaton simplifies to:

$$\sum \gamma_i Q_i \le \phi R_n = R_r$$

Where:

γ_i = Load Factor, statistically based multiplier applied to force effects — LRFD 1.3.2.1

Q_i = Force Effect (Moment or Shear)

ϕ = Resistance Factor

R_n = Nominal Resistance

R_r = Factored Resistance

Service limit state

Service limit state shall be taken as restriction on stress, deformation and crack width under regular service conditions. — LRFD 1.3.2.2

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Load combinations and load factors

The Total Factored Force Effect shall be taken as: | LFRD Eqn. 3.4.1-1

$$Q = \sum \gamma_i Q_i$$

Where:

γ_i = Load Factors specified in Tables 1 & 2

Q_i = Force Effects from loads specified in LRFD

Strength-I load combination relating to the normal vehicle use of the bridge without wind (See Section 11). | LRFD 3.4.1

Service-I load combination relating to the normal operational use of the bridge. | "

Service-III load combination relating only to tension in prestressed concrete structures with the objective of crack control. | "

$$Q_{Strength\text{-}I} = \gamma_{DC} DC + \gamma_{DW} DW + 1.75(LL + IM)$$ | LRFD Tables 3.4.1-1 & 2

$Q_{Service\text{-}I}$ = 1.0 (DC + DW) + 1.0 (LL + IM) | " "

$Q_{Service\text{-}III}$ = 1.0 (DC + DW) + 0.8 (LL + IM)

Force effects due to temperature, shrinkage, and creep because of the **free movement** at the end piers are considered to be zero.

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7 Vehicular Live Load | LRFD 3.6.1

Design live load | LRFD 3.6.1.2.2

Design live load designated as HL-93 shall be taken as:

LL = Truck or tandem (1 + IM) + Lane (See Item 2) | LRFD 3.6.1.2.1

Single Span Length =	45.00	ft			
HS-20 Truck Axles	32.00	32.00	8.00	kips	Section 2
HS-20 Truck Axle Spacing		14.00	14.00	ft	"
Tandem Truck Axles	25.00	25.00	kips		"
Tandem Truck Axle Spacing		4.00	ft		"
Lane load density, w_L	0.64	k/ft			LRFD 3.6.1.2.4

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Maximum live load force effect

Max Shear, V_{max}, occurs at the greater horizontal distance of d_v or $0.5d_v$cotq from the face of support where d_v is the effective depth between the tensile and compressive resultant forces in the member. Assume θ_{skew} is 45 degrees and face of support is the Centerline of bearing/pier (dowel connection). Use d_v to be conservative. | LRFD5.8.2.9 & LRFD5.8.3.2

Max Moment, Mmax, occurs near midspan (CL) underneath the nearest concentrated load (P1) when that load is the same distance to midspan as the center of gravity (+ CG) is to midspan. Use the Truck or Tandem (Near Midspan) and the Lane (At Midspan) maximum moments together to be conservative. | AISC LRFD Steel Manual, SS General Rules, p4-205 & BDM 4.3-B1

Truck: $V_{max(Truck)}$ = 54.57 kips At 0.72h | LRFD 5.8.3.2

$M_{max(Truck)}$ = 538.71 ft-kips Near midspan

AISC LRFD p4-205 & BDM 4.3-B1 & See Design LL Worksheet

Tandem: $V_{max(Tandem)}$ = 46.04 kips At 0.72h

$M_{max(Tandem)}$ = 513.61 ft-kips Near midspan

So the **HL-93 Live Load, LL = HS-20 Truck(1+IM)+HS-20 Lane** **Controls**

Lane: $V_{max(Lane)}$ = 13.40 kips At 0.72h

$M_{max(Lane)}$ = 162.00 ft-kips At midspan

Table 7-1: Live Load Reactions & Moments

Simple span	Reaction, kips	Moment, ft-kips	
	Support	0.72h	0.5 L
HS-20 Truck	57.1	85.1	538.7
HS-20 Lane	14.4	21.7	162.0

AASHTO HL-93 Truck 538.7 ft-kips **Truck LL Governs**

(This answer should check with AASHTO App. A Table)

AASHTO Lane load 162.0 ft-kips

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Dynamic load allowance (Impact, IM) LRFD 3.6.2.1

The static effect of Design Truck LL shall be increased by the following percentage : IM = 33% For bridge components (girder) LRFD Table 3.6.2.1-1

M(LL+IM)= 538.7 x(1+ 0.33)+ 162.0 = 878.5 ft-k

R(LL+IM)= 57.1 x(1+ 0.33)+ 14.4 = 90.3 k

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Distribution of Live Load, Df_i (Beam Slab Bridges) LRFD 4.6.2.2

For Multibeam deck bridges with conditions as follows, the approximate method of live load distribution applies with the following conditions : LRFD 4.6.2.2.1

Width of deck is constant

Number of Beams, $N_b \geq 4$

Beams are parallel

Beams have approximately the same stiffness

Roadway overhang, $d_e \leq 3.0\,ft$

Curvature in plane is less than 12 degree LRFD 4.6.1.2

X-section is one consistent with one listed in LRFD Table 4.6.2.2.1-1

The multiple presence factor shall not be applied in conjunction with approximate load distribution except for exterior beams. LRFD 3.6.1.1.2

The typical x-section **g** applies to voided and solid slabs w/o P.T. LRFD Table 4.6.2.2.1-1

Distribution Factor for Moment Interior Girder, DF_{MInt} LRFD 4.6.2.2.2b

For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies :

Range of applicability: Width of beam, 35 < b = 48 < 60 in — LRFD Table 4.6.2.2.2b-1

Span length, 20 < L = 45.00 < 120 ft

Number of Beams, 5 < N_b = 11 < 20

Regardless of the number of design Lane Loaded: " "

S/D = 0.35

$D1 = 11.5 - N_L + 1.4N_L(1-0.2C)^2 =$ 11.575 when C<=5

$D2 = 11.5 - N_L =$ 8.50 when C>5

D = 11.57 " "

Where : k = ((1+m)I/J)^0.5 = 0.74 " "

C = k(W/L) < k = 0.7

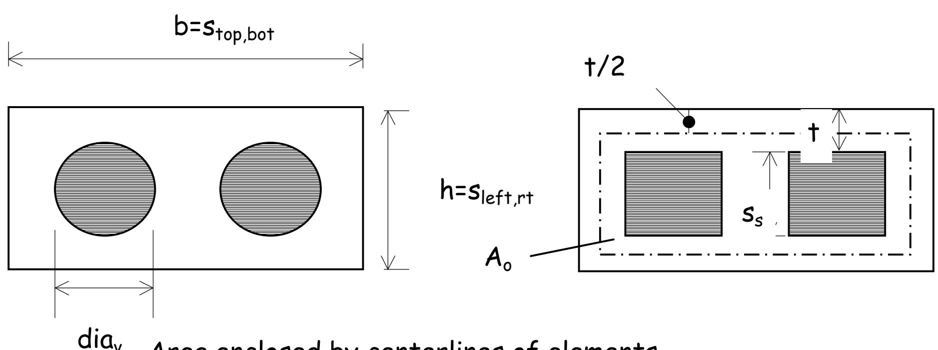
(Solid Slabs) I_P = $(d*b/12)*(b^2+d^2)$ = 309920 in^4

Calcs.Table 5-2

St Venant's Torsional Inertia — LRFD Eqn. C4.6.2.2.1-3

Voided Slab		Solid Slab	
$\boxed{J \approx \dfrac{4A_o^2}{\sum \frac{s}{t}} =}$	141596 in^4	$\boxed{J \approx \dfrac{A^4}{40.0 I_p} =}$	44292.3 in^4

J = 141596



Area enclosed by centerlines of elements

A_o = (h - t)(b - t) = 779 in^2

Thickness of plate-like element, t, is derived from converting the Void Area to Square Areas and centering the side vertically.

*t = $(h-s_s)/2$

where side of square, s_s, is derived from:

$A_{square} = A_{void}$

$$s_s = \sqrt{\pi\left(\frac{dia_v}{2}\right)^2} = \quad 14 \quad \text{in}$$

h = 26 in

t = 7.00 in

*Assume thickness, t, is constant throughout exterior elements, side elements, s, are exterior out-to-out girder measurements, b & h, interior element (leg between voids) is ignored.

Moment Distribution Factor for Interior girder, DF_{MInt}

One design Lane Loaded: 0.346

Skew Reduction Factor for Moments

LRFD 4.6.2.2.2e-1

Range of applicability: Skew, $0 < \theta_{skew}$ = 0.00 < 60°

Reduction Factor = 1.05 - 0.25 tan(θ) < 1.0

Reduction Factor = 1.000

Moment Distribution Factor for Skewed Interior girder,

DF_{MInt} = 0.346

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Moment Distribution Factor for Exterior girder, DF_{MExt}

LRFD 4.6.2.2.2d

For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies :

LRFD 4.6.2.2.1

LRFD 2.5.2.7.1

$DF_{MExt} = e \times DF_{MInt}$

Where Skew Reduction Factor is included in DF_{MInt} and Correction Factor

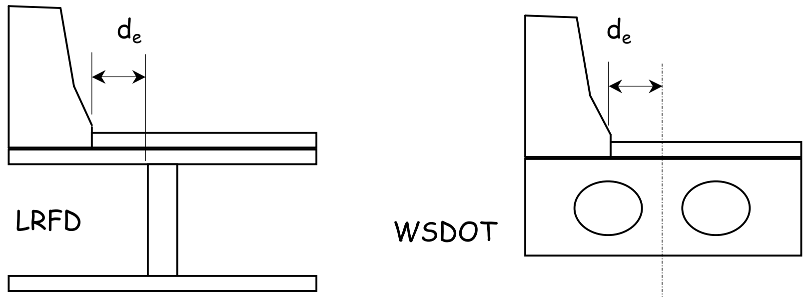
LRFD Table 4.6.2.2.2d-1

$$e = 1.04 + \frac{d_e}{25} \geq 1.0$$

Where the distance between outside face of exterior girder web to interior face of traffic barrier (LRFD overhang) is approximately equal to the distance from the centerline of the exterior girder to the inside face of traffic barrier (WSDOT)

BK

BK & RMP



CL

barrier footprint = 18.50 in

d_e = 0.46 ft ≤ 2 ft OK — LRFD Table 4.6.2.2.2d-1

e = 1.058 > 1.0

Moment Distribution Factor for Skewed Exterior Girder,

DF_{MExt} = 0.366

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Shear Distribution Factor for Interior Girder, DF_{VInt} — LRFD 4.6.2.2.3a

For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies : — LRFD Table 4.6.2.2.3a-1

Range of applicability: Width of beam, 35 < b =	48	< 60 in
Span length, 20 < L =	45.00	< 120 ft
Number of Beams, 5 < N_b =	11	< 20
St Venant Torsional Inertia, 25000 < J =	141596	< 610000 in^4
Net Moment of Inertia, 40000 < I_c =	64339	< 610000 in^4

By substituting the above pre-determined values, the approximate live load distribution factor for shear may be taken as the greater of:

One design Lane Loaded:

$$DF_{VInt} = \left(\frac{b}{130L}\right)^{0.15}\left(\frac{I_c}{J}\right)^{0.05} = \quad 0.468$$

LRFD Table 4.6.2.2.3a-1

Two or more Lanes Loaded:

$$DF_{VInt} = \left(\frac{b}{156}\right)^{0.4}\left(\frac{b}{12.0L}\right)^{0.1}\left(\frac{I_c}{J}\right)^{0.05} = \quad 0.471$$

" "

Skew Reduction Factor for Shear

LRFD Table 4.6.2.2.3c-1

Range of applicabilitySkew, $0 < \theta_{skew}$ =	0.00	< 60°
Span length, 20 < L =	45.00	< 120 ft
Depth of beam or stringer, 17 < d =	26	< 60 in
Width of beam, 35 < b =	48	< 60 in
Number of Beams, $5 < N_b$ =	11	< 20

$$RF_\theta = 1.0 + \frac{12.0L}{90d}\sqrt{\tan\theta} =$$ 1.000

Shear Distribution Factor for Skewed Interior Girder,

DF_{VInt} = 0.471

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Shear Distribution Factor for Skewed Exterior Girder

LRFD 4.6.2.2.3b

LRFD 4.6.2.2.1

LRFD 2.5.2.7.1

LRFD Table 4.6.2.2.3b-1

LRFD Table 4.6.2.2.3b-1

For Multibeam deck bridges within the range of applicability and conditions as follows, the approximate method of live load distribution applies :

Range of applicability:

Overhang, d_e = 0.46 < 2.0 ft

$DF_{VExt} = e \times DF_{VInt}$

where :

$$e = 1.02 + \frac{d_e}{50} \geq 1.0$$

e = 1.03 > 1.0

Shear Distribution Factor for Skewed Exterior Girder,

DF_{VExt} = 0.485

Table 7-2: Summary of Live Load Distribution Factors:

		Moment		Shear
Interior Girder	DF_{MInt} =	0.346	DF_{VInt} =	0.471
Exterior Girder	DF_{MExt} =	0.366	DF_{VExt} =	0.485

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8 Computation of Stresses

Sign convention: + Tensile stress
- Compressive stress

Concrete stresses due to Dead Load, DC + DW

Dead Load of structural components, DC, includes Girder & Traffic Barrier. LRFD 3.3.2

Stresses due to Weight of Girder (DC)

Unit weight, w_g = 0.96 k/ft BDM 6.4-A3-1

At d_v: x = 0.72 h = 1.56 ft LRFD 5.8.3.2

(V_{max} Location)

$$V_G = w_G\left(\frac{L}{2} - x\right) =$$ 20.03 kips AISC LRFD p 4-190

$$M_G = \frac{w_G x}{2}(L - x) =$$ 32.41 ft-kips " "

At mid-span x = 0.5 L = 22.5 ft

(M_{max} Location)

$$V_G = w_G\left(\frac{L}{2} - x\right) =$$ 0 kips " "

$$M_G = \frac{w_G L^2}{8} =$$ 242.1 ft-kips " "

Table 8-1: Stresses due to Girder Dead Load, σ_G

		0.72h	0.5 L
Top of girder	ksi	-0.079	-0.587
Bottom of girder	ksi	0.079	0.587

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Concrete stresses due to Traffic Barrier (DC)

Weight of Traffic Barrier over three girders, w_{TB3G} = $\frac{w_{TB}}{3} =$ 0.17 k/ft BDM 6.3.1B.2d

At d_v: x = 0.72 h = 1.56 ft

$$V_{TB} = w_{TB}\left(\frac{L}{2} - x\right) =$$ 3.49 kips AISC LRFD p 4-190

$$M_{TB} = \frac{w_{TB} x}{2}(L - x) =$$ 5.65 ft-kips " "

At mid-span x = 0.5 L = 22.50 ft

$$V_{TB} = w_{TB}\left(\frac{L}{2} - x\right) =$$ 0.00 kips " "

$$M_{TB} = \frac{w_{TB} L^2}{8} =$$ 42.19 ft-kips " "

Table 8-2: Stresses due to Traffic Barrier, σ_{TB}

		0.72h	0.5 L
Top of girder	ksi	-0.014	-0.102
Bottom of girder	ksi	0.014	0.102

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Concrete stresses due to SIDL, Asphalt Overlay (DW)

Weight of Asphalt Overlay, w_{SIDL} = 0.16 k/ft

At d_v: x = 0.72 h = 1.56 ft

$$V_{SIDL} = w_{SIDL}\left(\frac{L}{2} - x\right) = \quad 3.35 \text{ kips}$$ " "

$$M_{SIDL} = \frac{w_{SIDL}x}{2}(L - x) = \quad 5.42 \text{ ft-kips}$$ " "

At mid-span x = 0.5 L = 22.50 ft

$$V_{SIDL} = w_{SIDL}\left(\frac{L}{2} - x\right) = \quad 0.00 \text{ kips}$$ " "

$$M_{SIDL} = \frac{w_{SIDL}L^2}{8} = \quad 40.50 \text{ ft-kips}$$ " "

Table 8-3: Stresses due to SIDL, σ_{DW}

		0.72h	0.5 L
Top of girder	ksi	-0.013	-0.098
Bottom of girder	ksi	0.013	0.098

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Concrete stresses due to Live Load LRFD 3.6.1.2.1

Dynamic load allowance: IM = 33% LRFD 3.6.2.1

Table 8-4: Live Load Force Effect:

Simple span	Moment, ft-kips		Shear, kips	
	0.72h	0.5 L	0.72h	0.5 L
HS-20 Truck	85.1	538.71	54.6	6.13
HS-20 Lane	21.7	162.00	13.4	0.00

Table 8-5: HL-93 Live Load, LL = HS-20 Truck(1+IM)+HS-20 Lane

Simple span	Moment, ft-kips		Shear, kips	
	0.72h	0.5 L	0.72h	0.5 L
	134.91	878.486	85.98	8.16

Table 8-6: Distributed Live Load

Simple span	Moment, ft-kips		Shear, kips	
	0.72h	0.5 L	0.72h	0.5 L
Interior Girder	46.62	303.59	40.50	3.84
Exterior Girder	49.34	321.30	41.68	3.95

Table 8-7: Stresses in Girder due to LL+IM :

		0.72h	0.5 L
Top of Girder	ksi	-0.120	-0.779
Bottom of Girder	ksi	0.120	0.779

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Summary of stresses at d_v (Table 8-8)

Stresses, ksi	Top of girder	Bottom of girder
Weight of Girder	-0.079	0.079
Weight Traffic Barrier	-0.014	0.014
Weight of SIDL	-0.013	0.013
Live Load plus Impact Service - I	-0.120	-
Live Load plus Impact Service - III	-	0.096

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Summary of stresses at Mid-Span (Table 8-9)

Stresses, ksi	Top of girder	Bottom of girder
Weight of Girder	-0.587	0.587
Weight Traffic Barrier	-0.102	0.102
Weight of SIDL	-0.098	0.098
Live load plus impact Service - I	-0.779	-
Live load plus impact Service - III	-	0.623

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9 Approximate Evaluation of Pre-Stress Losses

For Prestress losses in members constructed and prestressed in a single stage, relative to the stress immediately before transfer, in pretensioned members, with low relaxation strands, the **Total Lump Sum Losses** may be taken as: LRFD 5.9.5.1

$$\Delta f_{pT} = \underbrace{\Delta f_{pSR} + \Delta f_{pCR} + \Delta f_{pR}} + \Delta f_{pES}$$

Time Dependent Losses

Where losses due to Shrinkage (SR), Cracking (CR), and Steel Relaxation (R), can be determined as a Lump Sum Estimate of Time Dependent Losses. LRFD 5.9.5.3

Time Dependent Losses

For normal weight concrete pretensioned by low-relaxation strands, approximate lump-sum time dependent losses resulting from creep and shrinkage of concrete and relaxation of prestressing steel may be used as follows:

For Voided slab with 270 ksi strands,

Time Dependent Losses = $37\left[1-0.15\frac{(f_c'-6)}{6}\right]=$ 36.08 ksi — BDM Table 6.1.5-1

Losses due to elastic shortening should be added to time-dependent losses to determine the total losses. — BDM 6.1.5B

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Loss due to elastic shortening — LRFD 5.9.5.2.3

The loss due to elastic shortening in pretensioned members shall be

$$\Delta f_{pES}=\left(\frac{E_p}{E_{ci}}\right)f_{cgp}$$

LRFD Eqn. 5.9.5.2.3a-1 & BDM 6.1.5B

Concrete strength at transfer:		f_{ci}' =	6.0	ksi
Modulus of elasticity,	concrete:	E_{ci} =	5173.3	ksi
	prestressing steel:	E_p =	28500.0	ksi

f_{cgp} = Stress due to prestressing and girder weight at Centroid of prestressing strands, at section of maximum moment

Eccentricities of Prestress Strands

C. G. of bottom strands to bottom of girder =	2.00	in.	BDM 6.6-A3-1
C. G. of top strands to bottom of girder =	23.00	in.	"
C. G. of bonded bottom strands to C.G. of girder, e_{bb}	11.00	in.	
C. G. of debonded strands to C.G. of girder, e_{db} =	11.00	in.	
C. G. of all bottom strands to C.G. of girder, e_b =	11.00	in.	"
C. G. of top strands to C.G. of girder, e_t =	-10.00	in.	"
E = C. G. of all strands to C.G. of girder =	**7.18**	**in.**	

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Concrete stress at Centroid of prestressing

$P_i = N\,A_{ps}\,f_p$ = 636.2 kips — BDM 6.1.5B

$$f_{ps}=-\frac{P_i}{A_g}-\frac{P_ie^2}{I_g}=$$ -1.25 ksi

$$f_g=\frac{M_ge}{I_g}=$$ 0.32 ksi

$f_{cgp} = f_g + f_{ps}$ = -0.92 ksi

Elastic shortening loss,

$$\Delta f_{pES} = \left(\frac{E_p}{E_{ci}}\right) f_{cgp} = \quad \mathbf{5.09} \quad \mathbf{ksi}$$

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Losses due to Steel Relaxation LRFD 5.9.5.4.4b

Steel Relaxation Loss at Transfer, LRFD Eqn. 5.9.5.4.4b-2

$$\Delta f_{pR1} = \frac{\log(24.0t)}{40.0}\left[\frac{f_{pj}}{f_{py}} - 0.55\right] f_{pj}$$

f_{py} = 0.90 f_{pu} = 243.00 ksi LRFD Table 5.4.4.1-1

Curing time for concrete to attain f'_{ci} is approximately 12 hours.
Tendons are stressed shortly before concrete pouring and prestressing forces transferred to concrete shortly after curing. An approximate time of one day is reasonable for steel relaxation loss calculation at transfer.

t = 1.00 day

$f_{pj`}$ = 0.80 f_{pu} = 216.0 ksi

f_{pj} = 0.75 f_{pu} + Δf_{pR1} = 205.0 ksi

Jacking Force

P_J = (f_{pj}) (A_{ps}) (N_{ps}) = 690.1 kips

P_J = (f_{pj}) (A_{ps}) (N_{ps}) = 564.6 kips Bottom Strands

By substitution :

Δf_{pR1} = 2.53 ksi

Above relaxation losses not added to Time Dependent Losses, but will be used for Service Limit State Total Transfer PS Losses, Section 10.

Total lump-sum losses,

Δf_{pT} = 38.64 ksi

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10 Stresses at Service Limit State

Concrete stresses at mid-span

Force per strand P / N = $A_{ps}(f_{pbt}-\Delta f_{pT})$ = 25.07 kips

Table 10-1: Mid-Span Prestress Force Effects

	No. of strands	Force per Strand, kips	Total force	Eccent. in.	Moment in-kip
Bottom Strands	14	25.07	350.98	11.00	3861
Debonded Strands	4	25.07	100.28	11.00	1103
Top Strands	4	25.07	100.28	-10.00	-1003
		P (kips)=	552	Mp(in-k) =	3961

Prestressing stress at top of girder

$$f_{p(top)} = -\frac{P}{A_c} + \frac{M_{ps}}{S_t} = \quad 0.16 \quad \text{ksi}$$

LRFD 5.9.4.2.1

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Final stress at top of girder

Stresses due to permanent loads plus prestressing :

$f_{g(topDL+PS)}$ = -0.63 ksi

Stresses due to all loads plus prestressing :

$f_{g(topDL+LL+PS)}$ = -1.41 ksi

Stresses due to Transient loads and one-half of permanent loads plus prestressing:

$f_{g[topLL+(1/2DL+PS)]}$ = -1.09 ksi

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Compressive stress limit at service - I load combinations

Due to permanent loads (DL + PS) :

$f_{comp.}$ =0.45 f'_c =	-3.15	ksi	>	-0.63	ksi	LRFD Table 5.9.4.2.1-1

Due to permanent loads and transient loads (DL + PS + LL):

$f_{comp.}$ = 0.60 f'_c =	-4.20	ksi	>	-1.41	ksi	"

Due to transient loads and one-half of permanent loads (LL + 1/2DL + 1/2PS):

$f_{comp.}$ = 0.40 f'_c =	-2.80	ksi	>	-1.09	ksi	"

Prestressing stress at bottom of girder

$$f_{p(bottom)} = -\frac{P}{A_c} - \frac{M_{ps}}{S_b} = \quad -1.441 \quad \text{ksi}$$

LRFD 5.9.4.2.2

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Final stress at bottom of girder

Stresses due to all loads plus prestressing :

$f_{g(bottom)}$ = -0.030 ksi

Tensile stress limit at service - III load combination

LRFD Table 5.9.4.2.2-1

$$f_{tens} = 0.0948\sqrt{f'_c} = \quad 0.251 \quad \text{ksi} \quad > \quad -0.030 \quad \text{ksi}$$

f_{tens} = 0.00 ksi > -0.030 ksi

per WSDOT office practice

BDM Table 6.2.3 2

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Stresses at transfer

The prestressing force may be assumed to vary linearly from zero at free end to a maximum at transfer length, l_t.

The transfer length may be taken as 60 times strand diameter. LRFD 5.8.2.3

$l_t = 60 \times d_{strand}/12 =$ 2.50 ft = 30.00 in. LRFD 5.11.4.1

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Determination of prestressing losses at transfer

At transfer the losses that could be accounted for are elastic shortening and steel relaxation only. (Back to Table of Contents)

LRFD 5.9.5.2.3a

5.9.5.4.4b

Total loss at transfer

$\Delta f_{pt} = \Delta f_{pES} + \Delta f_{pR1} =$ 7.62 ksi

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Concrete stress immediately after transfer at d_v from support

Prestressing forces at transfer = $N_{ps}A_{ps}(f_{pbt} - \Delta f_{pt})$

Force per strand, P/N = 30.2 kips

Table 10-2: Prestress Force Effects at Max. Shear Location, d_v

	No. of strands	Force k/stand	Total kips	Eccent. in.	Moment in-kips
Bottom Strands	14	30.20	422.8	11.00	4651
Debonded Strands	4	0.00	0.0	11.00	0
Top Strands	4	30.20	120.8	-10.00	-1208
		P_j =	543.7	M_{pj} =	3443

* At d_v some of the strands are Debonded

Stresses due to weight of girder :

$M_g =$ 32.41 ft-kips

Stresses: Top of girder, $\dfrac{M_g}{S_t} =$ -0.079 ksi

Bottom of girder, $\dfrac{M_g}{S_b} =$ 0.079 ksi

Prestressing stress at top of girder :

$$f_p = \frac{-P_j}{A_g} + \frac{M_{pj}}{S_t} = \quad 0.064 \text{ ksi}$$

Final stress at top of girder : $f_{g(top)} =$ -0.014 ksi

Tensile stress limit in areas without bonded reinforcement : LRFD 5.9.4.1.2

$$f_t = 0.0948\sqrt{f'_{ci}} \leq$$ 0.200 ksi > -0.014 ksi LRFD Table 5.9.4.1.2-1

Prestressing stress at bottom of girder :

$$f_p = \frac{-P_j}{A_g} - \frac{M_{pj}}{S_b} =$$ -1.33 ksi

Final stress at bottom of girder : $f_{g(bot)}$ = -1.25 ksi

Compressive stress limit in pretensioned components :

$f_{ci} = 0.60f'_{ci}$ = -3.60 ksi > -1.25 ksi LRFD 5.9.4.1.1

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LRFD 5.5.4.1

11 Strength Limit State

Strength limit state shall be considered to satisfy the requirements for strength and stability.

$$\eta \Sigma (\gamma i\, Qi) < \phi R_n = R_r$$ LRFD Eqn. 1.3.2.1-1

Resistance factor

ϕ =	0.90	Flexural in reinforced concrete	LRFD 5.5.4.2.1
ϕ =	1.00	Flexural in prestressed concrete	"
ϕ =	0.90	Shear	"
ϕ =	0.75	Axial Compression	

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Flexural forces LRFD 3.4.1

Strength - I load combination is to be considered for normal vehicular load without wind. LRFD TABLE 3.4.1-1

Load factors :

γ_{DC} =	1.25	Components and attachments (Girder + TB)
γ_{DW} =	1.50	Wearing surface (SIDL or ACP)
γ_{LL} =	1.75	Vehicular load (LL + Impact)

Flexural moment = 1.0 [1.25 DC + 1.5 DW + 1.75 (LL+IM)]

M_u = 978.4 ft.-kips

Checked using QConBridge program,

M_u =	**1836.0**	**ft.-kips**	**NG Mu No Check**

QConBridge 101-145MuChk.qcb Report p 4of 5

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Flexural resistance

For practical design an equivalent rectangular compressive stress distribution of 0.85 f'_c overall depth of $a = \beta_1 c$ may be considered. LRFD 5.7.3.1.1 & 5.7.2.2

β_1 = 0.70 for f'_c = 7.0 ksi

The average stress in prestressing strands, f_{ps}, may be taken as: | LRFD Eqn. 5.7.3.1.1-1

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right)$$

$$k = 2\left(1.04 - \frac{f_{py}}{f_{pu}}\right) = \quad 0.28$$

LRFD Table C5.7.3.1.1-1

Location of neutral axis :

For rectangular section without mild reinforcement :

$$c = \frac{A_{ps}f_{pu} + A_s f_y + A_s^{'} f_y^{'}}{0.85 f_c^{'} \beta_1 b + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

$A_s = A'_s = 0.0$ with no partial prestressing considered — LRFD 5.7.3.1.1

A_{ps} = 2.75 in.^{2} Area of prestressing strands

d_p = 24.00 in. Distance from extreme compression fiber to Centroid of prestressing strands.

By substitution : c1 = 3.565 in

OK for rectangular section

For T-section without mild reinforcement :

$$c = \frac{A_{ps}f_{pu} + A_s f_y + A_s^{'} f_y^{'} - 0.85 f_c^{`}(b - b_w)h_f}{0.85 f_c^{'} \beta_1 b_w + kA_{ps}\dfrac{f_{pu}}{d_p}}$$

b_w = 24.45 in

h_f = 7.00 in

c2 = 0.52 in

c = 3.565 in

$a = \beta_1 c$ = 2.50 in < t_f

Average stress in prestressing steel : — LRFD 5.4.4.1-1

$$f_{ps} = f_{pu}\left(1 - k\frac{c}{d_p}\right) = \quad 258.8 \text{ ksi.}$$

Tensile stress limit at strength limit state, f_{pu} = 270.0 ksi

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Limit for reinforcement LRFD 5.7.3.3

Maximum reinforcement LRFD 5.7.3.3.1

The maximum amount of prestressing and non-prestressing reinforcement shall be such that :

$$\frac{c}{d_e} \le 0.42$$ LRFD Eqn. 5.7.3.3.1-1

Where, $$d_e = \frac{A_{ps} f_{ps} d_p + A_s f_y d_s}{A_{ps} f_{ps} + A_s f_y}$$ LRFD Eqn. 5.7.3.3.1-2

Since A_s = 0.0

Then $d_e = d_p$ = 24.0 in.

$\frac{c}{d_e} =$ 0.149 < 0.42 **OK Not overreinforced**

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The nominal flexural resistance is primarily controlled by steel, however the behavior of the section is based on the ratio of c/de, Prestressed concrete members shall be designed so that steel is yielding as the ultimate capacity is de, Distance from extreme compression fiber to the Centroid of all tensile steel approached. This is satisfied by ductility limit requirement se as : c/de < 0.42. If the ratio of c/de > 0.42, section is considered over-reinforced and LRFD Equations C5.7.3.3.1-1&2 must LRFD C5.7.3.7.2

Nominal flexural resistance LRFD 5.7.3.2.3

Nominal flexural resistance of a rectangular section may be determined by using equations for flanged section in which case b_w shall be taken as b.

Under Reinforced Equations:

$a = \beta_1 c =$ 2.495 in.

Rectangular: $$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) =$$ 1351 ft.-kips

T-shaped: $$M_n = A_{ps} f_{ps}\left(d_p - \frac{a}{2}\right) + 0.85 f`_c (b - b_w)\beta_1 h_f (a/2 - h_f/2) =$$

1222 ft.-kips

M_n = 1351 ft.-kips

Over Reinforced Equations (applicable for Prestressed memebrs only):

Rectangular: $M_n = (0.36\beta_1 - 0.08\beta_1^{\,2}) f`_c b d_e^{\,2} =$ 3432 ft.-kips

T-shaped: $M_n = (0.36\beta_1 - 0.08\beta_1^{\,2}) f`_c b d_e^{\,2} + 0.85 f`_c (b - b_w) h_f (d_e - 0.5 h_f) =$

3424 ft.-kips

M_n = 3432 ft.-kips

Flexural resistance, $M_r = \phi\, M_n =$ 1351 > $M_u =$ 978 ft.-kips

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Minimum reinforcement LRFD 5.7.3.3.2

The amount of prestressing and non-prestressing steel shall be adequate to develop flexural resistance greater than or equal to the least 1.2 times the cracking moment or 1.33 times the factored moment required by Strength Limit State 1.

Flexural resistance,

$M_r = \phi M_n \geq$ The Lessor of: $1.2M_{cr}$ = 1027.5 ksi **governs**

$1.33M_u$ = 1301.3 ksi

$$M^*_{cr} = S_c\left(f_r + f_{pe}\right) - M_{d/nc}\left(\frac{S_c}{S_b} - 1\right)$$

1996 AASHTO 9.18.2.1

Beams are designed to be non-composite, therefore,
$S_c = S_b$ reduces the above equation to:

$$M^*_{cr} = S_b (f_r + f_{pe})$$

$f_r = 0.24\sqrt{f'_c} =$ 0.635 ksi LRFD 5.4.2.6

f_{pe} = -1.44 Stress at extreme fiber due to prestressing

M_{cr} = 856 ft.-kips

M_r = 1351 >1.2 Mcr = 1027.5 ft.-kips

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Development of prestressing strand

Pretensioning strand shall be bonded beyond the critical section for a development length taken as :

$$l_d \geq K\left(f_{ps} - \frac{2}{3} f_{pe}\right) d_b$$

K = 1.6

f_{ps} = 258.77 ksi

f_{pe} = 163.86 ksi

d_b = 0.50 in.

$l_d \geq$ 9.97 ft

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l_d = 9.97 ft < 1/2 Span   L/2 = 22.50   **OK developed**

5.11.4.2

LRFD Eqn. 5.11.4.2-1

12 Shear Design

Design procedure

The shear design of prestressed members shall be based on the general procedure of AASHTO - LRFD Bridge Design Specifications article 5.8.3.4.2 using the Modified Compression Field Theory.

Shear design for prestressed girder will follow the (replacement) flow chart for LRFD Figure C.5.8.3.4.2-5. This procedure eliminates the need for θ angle and β factor iterations.

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Effective Web Width, b_v, and Effective Shear Depth, d_v

Effective web width shall be taken as the minimum web width, measured parallel to the nuetral axis, between the resultants of the compressive and tensile forces due to flexure

b_v = Net Web = Width - Voids

b_v = 24.5 in.

Effective shear depth shall be taken as the distance between resultant of tensile and compressive forces due to flexure but it need not to be taken less than the greater of : 0.9de OR 0.72h.

$d_v = d_e - a/2$ = 22.8 in. **governs**

d_v = 0.9de = 21.6 in.

d_v = 0.72h = 18.7 in.

WSDOT Design Memo LRFD Shear Design, 3/19/02

WSDOT Design Memo, Shear Design 6/18/01

LRFD Figure C.5.8.3.4.2-5

LRFD 5.8.2.9

LRFD 5.8.3.2

The location of critical section for shear shall be taken as the larger of 0.5 dv cot(θ) or dv from the internal face of support.

Since θ , the angle of diagonal compressive stress, is not known at this point, use d_v for shear force calculation.

use d_v = 22.8 or 1.90 ft

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Component of Prestressing Force in Direction of Shear Force, V_p

LRFD Figure C.5.8.3.4.2-5

The prestressing in PCPS Slabs are horizontal only, there is no vertical component

V_p = 0.00 kips

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Shear Stress Ratio

LRFD Figure C.5.8.3.4.2-5

$$\frac{v_u}{f_c'} = \mathbf{0.0305}$$

Where the Shear Stress (ksi) on the concrete is,

LRFD Eqn. 5.8.2.9-1

$$v_u = \frac{V_u - \phi V_p}{\phi b_v d_v} = \quad 0.213 \text{ ksi}$$

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Where the

Factored shear force

$\mathbf{V_u = \Sigma(\eta_i \gamma_i V_i)}$

η_i =	1.00	Limit state factor for any ordinary structure
γ_{DC} =	1.25	Components and attachments (Girder + TB)
γ_{DW} =	1.50	Wearing surface (SIDL or ACP)
γ_{LL+IM} =	1.75	Vehicular load (LL + Impact)

Girder, V_g =	19.7	kips	
Traffic Barrier, V_{tb} =	3.4	kips	
V_{DC} =	23.1	kips	
ACP Overlay, V_{DW} =	3.3	kips	LRFD 3.6.1.2.1
$V_{LL+IM} \times DF_{VExt}$ =	41.7	kips	Approx. (from 0.72h)

Shear force effect,

$V_u = 1.00(1.25\ V_{DC} + 1.5\ V_{DW} + 1.75\ V_{LL+IM})$

$\mathbf{V_u}$ = **106.80 kips**

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f_{po}

LRFD Figure C.5.8.3.4.2-5

If the (critical) section (for shear) is within the transfer length of any (prestress) strands, calculate the effective value of f_{po}, the parameter taken as modulus of elasticity of prestressing tendons multiplied by the locked in difference in strain between the prestressing tendons and the surrounding concrete.

$f_{po} = 0.70 f_{pu}$ | LRFD Eqn. 5.8.3.4.2-3

$$f_{po} = \left[\frac{x + d_v}{l_t}\right] 0.70 f_{pu}$$

governs, dv is within the transfer length of the prestressed strands

Where the distance between the edge of girder (or beginning of prestress) and the CL of Bearing (BRG)

x = 5.00 in. | BDM 6.6-A3-1

accounting for bridge skew gives a long. distance from the face of girder as,

x = 5.00 in.

f_{po} = 174.84 ksi

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Longitudinal Strain (Flexural Tension), ε_x

The section contains at least the minimum transverse reinforcement as specified in Article 5.8.2.5. Longitudinal strain in the "web reinforcement" on the flexural tension side of the member,

WSDOT Design Memo Shear Design, 6/18/2001 & LRFD Eqn. 5.8.3.4.2-**1**

$$\varepsilon_x = \frac{\left[\frac{M_u}{d_v} + 0.5N_u + (V_u - V_p) - A_{ps} f_{po}\right]}{2(E_s A_s + E_p A_{ps})} \le 0.002$$

Where: **Factored moment** is not to be taken less than $V_u d_v$ | LRFD 5.8.3.4.2

$M_u = \Sigma (\eta_i \gamma i M_i)$

Ultimate moment at d_v from support, M_u

Girder, M_g =	39.1	ft-kips	
Traffice Barrier, M_{tb} =	6.8	ft-kips	
M_{DC} =	45.9	ft-kips	
ACP (SIDL), M_{DW} =	6.5	ft-kips	
$M_{LL+IM} \times DF_{MExt}$ =	49.3	ft-kips	Approx. (from 0.72h)

Moment Force Effect,

$M_u = 1.00(1.25M_{DC} + 1.50M_{DW} + 1.75M_{LL+IM})$

M_u = 153.5 ft-kips = 1842.3 in - kips

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Check which value governs:

$V_u d_v$ = 2430.1 in - kips **governs**

M_u = 1842.3 in - kips

Applied Factored Axial forces,

N_u = 0.00 kips

Factored Shear,

V_u = 106.80 kips

Vertical Component of Prestress Forces,

V_p = 0.00

Area of prestressing steel on the flexural tension side of the member,

$A_{ps(T)} = N_{bb} \times A_{ps}$ = 2.14 in.^{2}

Prestress/Concrete Modulus of Elasticity Parameter

f_{po} = 174.84 ksi

Modulus of Elasticity of Mild Reinforcement,

E_s = 29000 ksi

Area of Mild Reinforcement in flexural tension side of the member,

$A_{s(bottom)} = n_{s(bottom)} A_s$

Where there are 4 No. 4 bars | BDM 6.6-A3-1

$A_{s(bottom)}$ = 0.80 in.^{2}

Modulus of Elasticity of Prestress Strands,

E_p = 28500 ksi

Substitution gives,

ε_x = -0.0009549 **< 0, so use the following Equation 3:**

If the value of ε_x from LRFD Equations 5.8.3.4.2-1 or 2 is negative, the strain shall be taken as:

WSDOT Design Memo Shear Design, 6/18/2001 & LRFD Eqn. 5.8.3.4.2-**3**

$$\varepsilon_x = \frac{\left[\frac{M_u}{d_v} + 0.5N_u + (V_u - V_p) - A_{ps} f_{po}\right]}{2(E_c A_c + E_s A_s + E_p A_{ps})}$$

Where: Modulus of Elasticity of Concrete,

E_c = 5587.8 ksi

Area of concrete on the flexural tension side of the member,

A_c = 430.41 in.^{2}

Substitution gives,

ε_x = -0.0000323 **Equation 3 Governs**

ε_x = -0.0000323

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Determination of β and θ

Shear Stress Ratio of:	0.030	Is a value just ≤	0.075
1000 x the Long. Strain:	-0.032	Is a value just ≤	0.00

From Table 1: **θ = 21.80 deg.**

β = 3.75

LRFD Table 5.8.3.4.2-1 & See Theta and Beta Worksheet

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Shear strength | LRFD 5.8.3.3

$V_r = \phi V_n$

Nominal shear strength shall be taken as:

$V_n = V_c + V_s + V_p$

Shear resistance provided by concrete :

$$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v$$

Shear taken by shear reinforcements :

$V_s = V_n - V_c - V_p$ | LRFD 5.5.4.2

$\phi =$ 0.90 for shear

$V_n =$ Nominal shear strength | LRFD 5.8.3.4

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Required shear strength | LRFD 5.8.3.3

Nominal shear strength shall be taken as the lesser of :

$V_n = V_c + V_s i + V_p =$ 288.2 kips **governs** | LRFD Eqn. 5.8.3.3-1

$V_n = 0.25f'_c b_v d_v + V_p =$ 973.5 kips | LRFD Eqn. 5.8.3.3-2

Shear resistance provided by concrete :

$V_c = 0.0316\beta\sqrt{f'_c}b_v d_v =$ 174.4 kips | LRFD Eqn. 5.8.3.3-3

Shear taken by shear reinforcement: shear reinforcing not based on capacity

$V_{sreq} = V_u/ø - V_c - V_p =$ -55.7 kips | LRFD Eqn. 5.8.3.3-1

Spacing of shear reinforcements :

Try 2 legs of # 4 Av = 0.40 in.^{2}

Required Spacing, $s_{req'd} = \dfrac{A_v f_y d_v \cot\theta}{V_s} =$ -24.49 in. | LRFD Eqn. 5.8.3.3-4

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Maximum spacing of shear reinforcement | LRFD 5.8.2.7

if $v_u < 0.125\ f'_c$ then $s_{max} = 0.8\ dv <$ ~~24 in.~~ 18 in. | Eqn. 5.8.2.7-1

Maximum spacing of shear reinforcement, WSDOT Practice = 18.00 in | BDM 6.2.3.E.3

if $v_u >= 0.125\ f'c$ then $s < 0.4\ dv < 12$ in. | Eqn. 5.8.2.7-2

$v_u =$ 0.213 ksi

0.125 f'c = 0.875 ksi > $v_u =$ 0.213 ksi

Maximum spacing, s_{max} =	**18.0**	**in.**	**OK**
Governing spacing, s_{gov} =	**18.0**	**in.**	

$A_{v(provided)} =$ 0.40 in.^{2} Assuming two #4 legs

$$V_s = \frac{A_v f_y d_v \cot\theta}{s_{gov}} = \quad 75.85 \quad \text{kips}$$

LRFD Eqn. 5.8.3.3-4

Shear reinforcement is required if : $0.5\phi(V_c+V_p) < V_u$

$0.5\phi(V_c+V_p) =$ 78.5 < Vu = 106.8 kips

LRFD Eqn. 5.8.2.4-1

Yes, Shear/Transverse Reinf. Is Required

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Minimum shear reinforcement

LRFD 5.8.2.5

When shear reinforcement is required by design, the area of steel provided,

$$A_{v(provided)} \geq 0.0316\sqrt{f_c'}\frac{b_v s_{gov}}{f_y}$$

Use Spacing: 10.00 in <= 18.0 in

OK

LRFD Eqn. 5.8.2.5-1

where :

s = 10.0 in.

Required Area of Steel,

$$0.0316\sqrt{f_c'}\frac{b_v s_{gov}}{f_y} = \quad 0.34 \quad \text{in.}^2$$

0.40 > 0.34 in.^{2}

OK for Min. Transverse Reinf.

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Longitudinal reinforcement

LRFD 5.8.3.5

Longitudinal reinforcement shall be provided so that at each section the following equations are satisfied :

$$A_s f_y + A_{ps} f_{ps}\left(\frac{d_v}{l_t}\right) \geq T = \frac{M_u}{d_v \phi} + \frac{0.5N_u}{\phi} + \left(\frac{V_u}{\phi} - 0.5V_s - V_p\right)\cot\theta$$

LRFD Eqn. 5.8.3.5-1

BDM 6.6-A3-1

A_s =	0.80	in^2	
f_y =	60.00	ksi	
A_{ps} =	2.14	in^2	Area of prestressing steel on the flexural tension side of the member(w/o unbonded)
f_{ps} =	258.77	ksi	f_{ps} multiplied by d_v/l_t ratio to account for lack of prestress development
d_v =	22.75	in.	
l_t =	2.50	ft =	30.00 in
M_u =	202.5	ft-kips	
ϕ =	1.00		Flexural in prestressed concrete
ϕ =	0.90		Shear
ϕ =	0.75		Axial Compression
N_u =	0.00		

V_u = 106.80 kips
V_s = 75.85 kips
V_p = 0.00 kips
θ = 21.80 degree

by substitution:

468.38 ≥ 308.7 kips **OK for Longitudinal Reinforcement**

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13 Deflection and Camber

LRFD 5.7.3.6.2

Let downward Deflection be Positive +

Let upward Deflection, Camber, be Negative -

Deflection and camber calculations should be based on modulus of elasticity and maturity of concrete.

Instantaneous deflection should be computed for the effect of all dead loads and prestressing forces :

At 28 days:	f'_c =	7.0	ksi
	E_c =	5587.8	ksi
At final (120 days):	1.0 f'_c =	7.0	ksi
	E_c(1.0 f'_c) =	5587.8	ksi
At release:	f'_{ci} =	6.0	ksi
	E_{ci} =	5173.3	ksi

Deflection due to prestressing forces at Transfer

Deflection due to bottom strands is computed from a combination of fully bonded strands and the partially bonded or "debonded" strands which are sleeved at the ends of the girder. Each type has their own eccentricity.

$$\Delta ps_{bot.} = \Delta ps_{bb} + \Delta ps_{db}$$

$$\Delta ps_{bb} + \Delta ps_{db} = \left(P_{bb}e_{bb} + k_{db}P_{db}e_{db}\right)\frac{L^2}{8E_{ci}I_c}$$

Force and eccentricity due to the bonded bottom prestress strands are:

P_{bb} = 422.84 kips e_{bb} = 11.00 in

Reduction factor for the partially bonded or debonded strands

$$k_{db} = \frac{L - 2l_{db}}{L} = \quad 0.822$$

RMP

The average sleeved length of the debonded strands,

l_{db} = 4.0 ft = 48.0 in

BDM 6.6-A3-1

Force and eccentricity due to the debonded bottom prestress strands are:

P_{db} = 120.81 kips e_{db} = 11.00 in

$\Delta ps_{bot.}$ = -0.653 in. upward

Deflection due to top strands is computed from :

$$\Delta ps_{top} = \frac{P_t e_t L^2}{8E_{ci} I_c}$$

Prestressing force and eccentricity of top strands.

P_t = 120.8 kips e_t = 10.00 in.

Δps_{top} = 0.137 in. downward

Total deflection due to prestressing :

$\Sigma \Delta_{ps}$ = -0.65 + 0.14 = -0.515 in. upward

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Deflection due to weight of Girder

$$\Delta_g = \frac{5w_g L^4}{384 E_{ci} I_c} =$$ 0.27 in. downward

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Deflection due to weight of Traffic Barrier TB

$$\Delta_{tb} = \frac{5w_{tb(3G)} L^4}{384 E_c I_c}$$

Δ_{tb} = 0.04 in. downward

0.00 in.

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Deflection due to weight of Wearing Surface SIDL

$$\Delta_{SIDL} = \frac{5w_{SIDL} L^4}{384 E_c I_c}$$

Δ_{SIDL} = 0.04 in. downward

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Deflection (Camber) at transfer, C_i

Deflection accounted at transfer are due to prestressing and weight of girder :

At transfer : $\Sigma \Delta_i$ = -0.52 + 0.27 = -0.25 in.

$C_i = -\Delta_i$ = 0.25 in.

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Final deflection due to all loads, C_F and C_{F+SIDL}

Long term deflection may be taken as four times the instantaneous deflection if calculations are based on the gross moment of inertia, Ig. — LRFD 5.7.3.6.2

Long term deflection :

$4\,\Sigma\,\Delta$ = 4 (-0.52 + 0.27)= -1.00 in.

Using:

$\Sigma\,\Delta$ (long-term) = $\Sigma\,\Delta$ (elastic) X (Ψ (t, ti) + 1) — WSDOT

PS & Girder Long term Deflection

Find the Long term deflection (at 2000 days) due to Prestressing forces and Girder Self Weight only, C_F.

$$C_F = -\left[\left(\Delta_{PS} + \Delta_g\right)\left(\Psi_{(t,t_i)} + 1\right)\right]$$

Creep Coefficient : — LRFD 5.4.2.3.2

$$\Psi_{(t,t_i)} = 3.5 k_c k_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \frac{(t - t_i)^{0.6}}{10.0 + (t - t_i)^{0.6}}$$

LRFD Eqn. 5.4.2.3.2-1

Factor for the effect of concrete strength,

$$k_f = \frac{1}{0.67 + \left(\frac{f_c'}{9}\right)} = \quad 0.69$$

LRFD Eqn. 5.4.2.3.2-2

Factor for the effect of Volume-to-Surface ratio of the component specified in LRFD Figure 5.4.2.3.2-1,

$$k_c = \left[\frac{\dfrac{t}{26e^{0.36(V/S)} + t}}{\dfrac{t}{45 + t}}\right]\left[\frac{1.80 + 1.77e^{-0.54(V/S)}}{2.587}\right]$$

LRFD (Eqn.) C5.4.2.3.2.-1

$k_c(t)$ = 0.70

$k_c(t_i)$ = 0.23

$k_c(t-t_i)$ = 0.70

Assume 1 Day of accelerated cure by radiant heat or steam. 1 Day accelerated cure = 7 normal Days of cure. Age of Concrete when load is initially applied, — LRFD 5.4.2.3.2

t_i = 7 DAY

Maturity of Concrete,

t = 2000 DAY

B.K.

Volume-to-Surface Ratio,

$V/S =$ 5.02

Volume = $lhb - Vol_{voids}$

Actual length of the entire girder (endface-to-endface)

$l =$ 550.00 in.

Volume of Voids

$Vol_{Voids} = A_{Voids} l_{Void}$

End of Void is minimum distance of **15** in. from end of girder. BDM 6.6-A2-1

$l_{Void} =$ 520.00 in.

$Vol_{Voids} =$ 201337 in^3

Volume = 485063 in^3

Assume the voids are exposed to atmospheric drying, but with poor ventilation with small drain holes on each end. Therefore use 50% of Void surface area. Email 2

LRFD 5.4.2.3.2

Surface Area = $A_{ends} + A_{sides} + A_{top\&bot} + 1/2A_{voids}$

Area of ends that will account for Skew Angle if any,

$A_{ends} =$ 2496 in^2

$A_{sides} =$ 28600 in^2

$A_{top\&bot} =$ 52800 in^2

$1/2A_{voids} =$ 12824 in^2

Surface Area = 96720 in^2

H = **80** average humidity — LRFD Fig. 5.4.2.3.3-1

by substitution :

Ψ (t,ti)= 1.12

$\Sigma\Delta$ (long-term) = $\Sigma\Delta$ (elastic) X (Ψ (t, ti) + 1) = -0.53 in. upward

$$C_F = -[(\Delta_{PS} + \Delta_g)(\Psi_{(t,t_i)} + 1)] =$$ 0.53 in. upward

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Find the deflection (at 120 days) due to Prestressing forces, Girder Self Weight, $C_{120days}$.

$$C_{120\ days} = C_F - [(\Delta_{tb} + \Delta_{SIDL})(\Psi_{(t,t_i)} + 1)]$$

Creep Coefficient for TB and ACP from 7 to 120 days:

$$\Psi_{(t,t_i)} = 3.5k_c k_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \frac{(t-t_i)^{0.6}}{10.0 + (t-t_i)^{0.6}}$$

$k_f =$ 0.69

Age of Concrete when load is applied,

t_i = 7 DAY

t = 120 DAY

$$k_c = \left[\frac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}} \right] \left[\frac{1.80+1.77e^{-0.54(V/S)}}{2.587} \right]$$

Volume = 485063 in^3

Surface Area = 96720 in^2

V/S = 5.02

kc(ti) = 0.23

kc(t) = 0.44

kc(t-ti) = 0.43

H = 80 average humidity

by substitution :

Ψ (t,ti)= 0.48

Σ Δ (long-term) = Σ Δ (elastic) X (Ψ (t, ti) + 1) = -0.37 in. downward

Find the Long term deflection (at 2000 days) due to Prestressing forces, Girder Self Weight, Traffic Barrier, and ACP (SIDL), C_{F+SIDL}.

$$C_{F+SIDL} = C_F - [(\Delta_{tb} + \Delta_{SIDL})(\Psi_{(t,t_i)} + 1)]$$

Creep Coefficient for TB and ACP from 120 to 2000 days:

LRFD 5.7.3.6.2

LRFD 5.4.2.3.2

$$\Psi_{(t,t_i)} = 3.5 k_c k_f \left(1.58 - \frac{H}{120}\right) t_i^{-0.118} \frac{(t-t_i)^{0.6}}{10.0 + (t-t_i)^{0.6}}$$

k_f = 0.69

LRFD Fig. 5.4.2.3.2-1

B.K.

Age of Concrete when load is applied,

t_i = 120 DAY

t = 2000 DAY

$$k_c = \left[\frac{\dfrac{t}{26e^{0.36(V/S)}+t}}{\dfrac{t}{45+t}} \right] \left[\frac{1.80+1.77e^{-0.54(V/S)}}{2.587} \right]$$

Volume = 485063 in^3

Surface Area = 96720 in^2

V/S = 5.02

kc(ti) = 0.44

kc(t) = 0.70

kc(t-ti) = 0.70

H = 80 average humidity

by substitution :

Ψ (t,ti)= 0.79

$\Sigma\,\Delta$ (long-term) = $\Sigma\,\Delta$ (elastic) X (Ψ (t, ti) + 1) = 0.07 in. downward

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$$C_{F+SIDL} = C_F - [(\Delta_{tb} + \Delta_{SIDL})(\Psi_{(t,t_i)} + 1)] =$$ 0.46 in. upward

Camber Summary

C_i =	0.25	in.	Upward	P/S + slab DL
$C_{120days}$ =	-0.37	in.	Upward	P/S + slab DL at 120 days
C_F =	0.53	in.	Upward	P/S + slab DL at 2000 days
C_{F+SIDL} =	0.46	in.	Upward	P/S + slab DL+TB+ ACP Overlay

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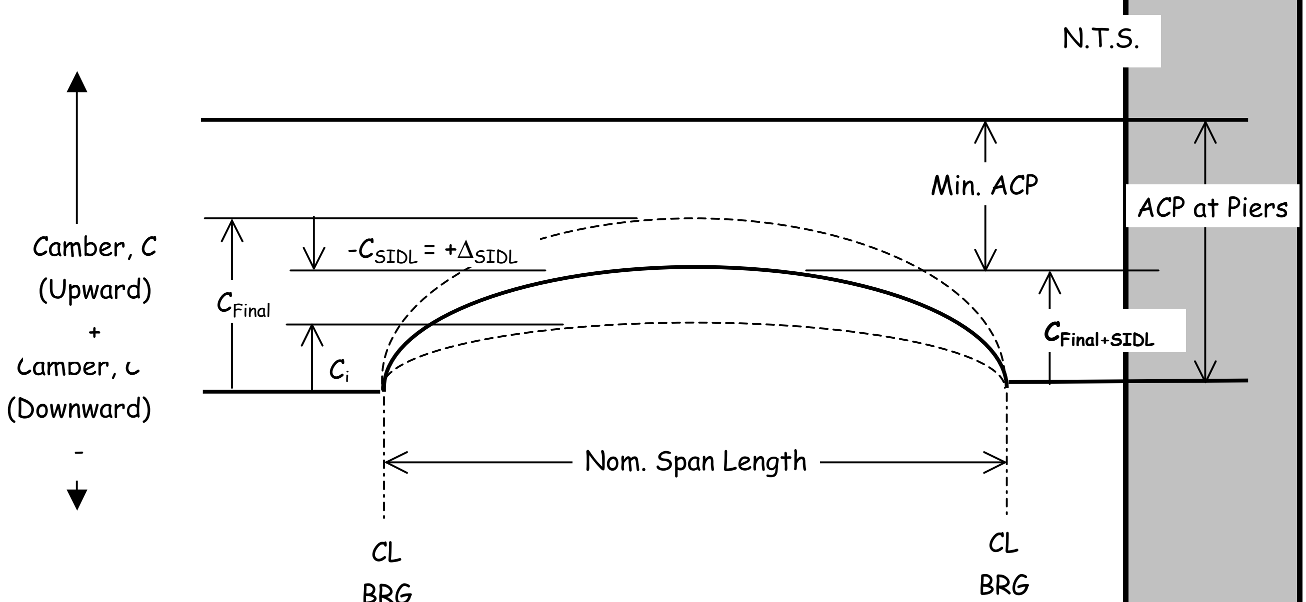

ACP at Piers

ACP at Piers = **3.46** in.

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References

AISC	LRFD Manual of Steel Construction (equation reference)
BDM	WSDOT Bridge Design Manual, August 2002
B.K.	WSDOT Bijan Khaleghi Concrete Design Specialist, 2002-2003
Email 1	Mel Hitzke Olympic Region Materials Engineer, 6/19/03
Email 2	Steve Seguirant of Concrete Tech, 6/04/03
LRFD	AASHTO LRFD Bridge Design Specifications, 2nd Edition 1998 with Interims thru 2002
Nilson	Design of PS Concrete, Author H. Nilson, 1987
RMP	WSDOT Richard Pawelka, 2002-2003
Prelim.Plan	West Fork Hoquiam Bridge Preliminary Plan, 2002-2003
Section	A Section Numbered in this Spreadsheet's Table of Contents
WSDOT	Washington State Dept. of Transportation Bridge & Structures office policy or memo

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Design Example 6 Positive EQ Reinforcement at Interior Pier of a Prestressed Girder

Bridge Geometry and Material Properties

Girder type	W74G	
$Sg := 7$	ft	Girder Spacing
$Sc := 20$	ft	Column Spacing
$Ng := 4$		Number of Girders
$Nc := 1$		Number of Columns
$es := 86$	in	
$eg := 42$	in	
$ds := 28$	in	
$Skew\_angle := 30$	degrees	
$Aps := .215$	in^2	per strand
$fps := 270$		ksi
$fy := 60$	ksi	

$Skew\_angle := Skew\_angle \cdot \frac{\pi}{180}$ $Skew\_angle = 0.524$ Radians

Top of the column Forces (from GTRSTRUDL):

Platic Moment, M_p (ft-kips)	$Mp := 12186$
Plastic Shear, V_p (kips)	$Vp := 1200$
Elastic Moment (R = 1), M_{eq} (ft-kips)	$Meq := 12167$
Elastic Shear, V_{eq} (kips)	$Veq := 1157$
Moment due to SIDL, M_{SIDL} (ft-kips)	$Msidl := 410$

Connection design forces and cg of superstructure

1.2 x (The lesser of plastic hinging or elastic EQ Forces)

Mc = 1.2[Min (Mp or Meq) + Min(Vp or Veq)*es] - M_{SIDL}

$$Mc := 1.2 \cdot \left(Meq + Veq \cdot \frac{es}{12} \right) - Msidl$$

$Mc = 24140.6$ (ft-kips)

Vc = 1.2[Min(Vp or Veq)]

$Vc := 1.2 \cdot (Veq)$

$Vc = 1388.4$ (kips)

Distributed forces to each girder @ cg of superstructure

DF = 0.5 if $L_1 = L_2$

DF= 0.67 if $L_1 = 2 \times L_2$

Where L1 and L2 are the lengths of the Spans that are Supported by the girder

Since L_1=L_2, $DF := 0.5$

$Mg := DF \cdot Mc \cdot \frac{Nc}{Ng}$ $\qquad Mg = 3017.58$ (ft-kips)

Bottom Strand Extension:

$Aps\_total := \frac{Mg \cdot 12}{0.9 \cdot fps \cdot eg}$ $\qquad Aps\_total = 3.548$ in^2

$Number\_of\_strands := \frac{Aps\_total}{Aps}$ $\qquad Number\_of\_strands = 16.5$

Use 17 strands for bottom strand extension.

Vertical Diaphragm Stirrups

$$Av := \frac{Mg}{\left[\left(\frac{Sg}{\cos(Skew\_angle)}\right) \cdot 0.9 \cdot fy \cdot \frac{ds}{12}\right]} \qquad Av = 2.963 \quad in^2/ft$$

$\mu := 1$ Coefficient of friction of normal weight concrete cast against hardened concrete

$$Avf := \frac{Vc}{(2 \cdot Sg \cdot \cos(Skew\_angle) \cdot fy \cdot \mu)} \qquad Avf = 1.909 \; in^2/ft \text{ each side}$$

Design Example : Positive Moment Connection for P/S Girder Bridges

Distance from Neutral axis of Girder to bottom strand of girder, $d := 38$ (in)

$eg := 6.5$ (ft)

Total Loads for the entire structure

$Mp := 12186$ (kip-ft)

$Vp := 1157$ (kips)

Per Girder Forces

$Mg := \frac{Mp}{Ng}$ $\qquad Mg = 3046.5$ (kip-ft)

$Vg := \frac{Vp}{Ng}$ $\qquad Vg = 289.25$ (kips)

Moment at Centroid of Crossbeam

$M := Mg + Vg \cdot eg$ $\qquad M = 4926.63$ (kip-ft)

Area of Steel for Strand Extension

$$Aps\_total := \frac{M \cdot 12}{0.9 \cdot fps \cdot d \cdot 2}$$

$Aps\_total = 3.201$ in^2

$$Number\_of\_strands := \frac{Aps\_total}{Aps}$$ $Number\_of\_strands = 15$

Moment for Fixity Reinforcment

$$Mf := (Mp + Vp \cdot eg) \cdot \frac{Sc}{Sg}$$

$Mf = 5.63 \times 10^4$ (kip-ft)

Spacing, $S := 5$ (inches)

$$Aps\_totalF := \frac{Mf \cdot 12}{0.9 \cdot fy \cdot S}$$

$Aps\_totalF = 2502.41$ (in^2)

Design Example 7 Noise Wall Type D-2K (Precast Panel on Shaft)

This design is based upon:

- AASHTO Guide Specifications for Structural Design of Sound Barriers - 1989 (including 2002 interim)
- AASHTO Standard Specifications for Highway Bridges 17th Ed. - 2002
- USS Steel Sheet Piling Design Manual - July 1984
- WSDOT Bridge Design Manual
- Caltrans Trenching and Shoring Manual - June 1995

This design doesn't account for the loads of a combined retaining wall / noisewall. A maximum of 2 ft of retained fill above the final ground line is suggested.

Concrete Properties:

$w_c := 160 \cdot pcf$ BDM 4.1.1

$f'c := 4000 \cdot psi$

$E_c := \left(\frac{w_c}{pcf}\right)^{1.5} \cdot 33 \cdot \sqrt{\frac{f'c}{psi}} \cdot psi$ $E_c = 4.224 \times 10^6\,psi$ Std Spec. 8.7.1

$\beta_1 := \text{if}\left(f'c \le 4000 \cdot psi, 0.85, \max\left(0.85 - \frac{f'c - 4000 \cdot psi}{1000 \cdot psi} \cdot 0.05, 0.65\right)\right)$ Std Spec. 8.16.2.7 $\beta_1 = 0.85$

$f_r := 7.5 \cdot \sqrt{\frac{f'c}{psi}} \cdot psi$ $f_r = 474.3\,psi$ Std Spec. 8.15.2.1.1

Reinforcement Properties:

Diameters: $dia(bar) := \begin{cases} 0.375 \cdot in & \text{if } bar = 3 \\ 0.500 \cdot in & \text{if } bar = 4 \\ 0.625 \cdot in & \text{if } bar = 5 \\ 0.750 \cdot in & \text{if } bar = 6 \\ 0.875 \cdot in & \text{if } bar = 7 \\ 1.000 \cdot in & \text{if } bar = 8 \\ 1.128 \cdot in & \text{if } bar = 9 \\ 1.270 \cdot in & \text{if } bar = 10 \\ 1.410 \cdot in & \text{if } bar = 11 \\ 1.693 \cdot in & \text{if } bar = 14 \\ 2.257 \cdot in & \text{if } bar = 18 \end{cases}$

Areas: $A_b(bar) := \begin{cases} 0.11 \cdot in^2 & \text{if } bar = 3 \\ 0.20 \cdot in^2 & \text{if } bar = 4 \\ 0.31 \cdot in^2 & \text{if } bar = 5 \\ 0.44 \cdot in^2 & \text{if } bar = 6 \\ 0.60 \cdot in^2 & \text{if } bar = 7 \\ 0.79 \cdot in^2 & \text{if } bar = 8 \\ 1.00 \cdot in^2 & \text{if } bar = 9 \\ 1.27 \cdot in^2 & \text{if } bar = 10 \\ 1.56 \cdot in^2 & \text{if } bar = 11 \\ 2.25 \cdot in^2 & \text{if } bar = 14 \\ 4.00 \cdot in^2 & \text{if } bar = 18 \end{cases}$

$f_y := 60000 \cdot psi$

$E_s := 29000000 \cdot psi$ Std. Spec. 8.7.2

Wall Geometry:

Wall Height:	$H := 24 \cdot ft$	H should be <= 28 ft
Half of Wall Height:	$h := H \cdot 0.5$	$h = 12\,ft$
Shaft Diameter:	$b := 2.50 \cdot ft$	
Shaft Spacing:	$L := 12 \cdot ft$	

Wind Load (Guide Spec. Table 1-2.1.2.C):

$WindExp := \text{"B2"}$ Wind Exposure B1 or B2 - Provided by the Region

$WindVel := 90 \cdot mph$ Wind Velocity 80 or 90 mph - Provided by the Region

$$WindPressure(WindExp, WindVel) := \begin{vmatrix} 12 \cdot psf & \text{if } (WindExp = \text{"B1"} \wedge WindVel = 80 \cdot mph) \\ 16 \cdot psf & \text{if } (WindExp = \text{"B1"} \wedge WindVel = 90 \cdot mph) \\ 20 \cdot psf & \text{if } (WindExp = \text{"B2"} \wedge WindVel = 80 \cdot mph) \\ 25 \cdot psf & \text{if } (WindExp = \text{"B2"} \wedge WindVel = 90 \cdot mph) \\ \text{"error"} & \text{otherwise} \end{vmatrix}$$

Wind Pressure: $P_w := WindPressure(WindExp, WindVel)$ $P_w = 25\,psf$

Seismic Load (Guide Spec. 1-2.1.3):

Acceleration Coefficient	$A := 0.29$	BDM 4.4-A2
DL Coefficient, Wall	$f := 0.75$	Not on bridge condition
Panel Plan Area:	$A_{pp} := 4in \cdot L + 13in \cdot 16in$	$A_{pp} = 5.44\,ft^2$
Seismic Force EQD (perp. to wall surface):	$EQD := \max(A \cdot f, 0.1) \cdot \left(\frac{A_{pp} \cdot w_c}{L}\right)$	$EQD = 15.8\,psf$

Factored Loads (Guide Spec. 1-2.2.2):

$Wind := 1.3 \cdot P_w \cdot 2 \cdot h \cdot L$	$Wind = 9360\,lbf$	
$EQ := 1.3 \cdot EQD \cdot 2 \cdot h \cdot L$	$EQ = 5911\,lbf$	
$P := \max(Wind, EQ)$	$P = 9360\,lbf$	Factored Design load acting at mid height of wall "h".

Soil Parameters:

Soil Friction Angle:	$\phi := 38 \cdot deg$		Provided by the Region
Soil Unit Weight:	$\gamma := 125 \cdot pcf$		Provided by the Region
Top Soil Depth:	$y := 2.0 \cdot ft$		From top of shaft to ground line
Ineffective Shaft Depth:	$d_o := 0.5 \cdot ft$		Depth of neglected soil at shaft
Isolation Factor for Shafts:	$Iso := \min\left(3.0, 0.08 \cdot \frac{\phi}{deg}, \frac{L}{b}\right)$ $Iso = 3.00$		Factor used to amplify the passive resistance based on soil wedge behavior resulting from shaft spacing - Caltrans pg 10-2.
Factor of Safety:	$FS := 1.00$		
Angle of Wall Friction:	$\delta := \frac{2}{3} \cdot \phi$	$\delta = 25.333\,deg$	Guide Spec. App. C pg. 33
Correction Factor for Horizontal Component of Earth Pressure:	$HC := \cos(\delta)$	$HC = 0.904$	
Foundation Strength Reduction Factors:	$\phi_{fa} := 1.00$	(Active)	Guide Spec. 1-2.2.3
	$\phi_{fp} := 0.90$	(Passive)	Guide Spec. 1-2.2.3

REDUCTION FACTOR (R) OF K_p FOR VARIOUS RATIOS OF $-\delta/\phi$

ϕ \ δ/ϕ	−0.7	−0.6	−0.5	−0.4	−0.3	−0.2	−0.1	−0.0
10	.978	.962	.946	.929	.912	.898	.881	.864
15	.961	.934	.907	.881	.854	.830	.803	.775
20	.939	.901	.862	.824	.787	.752	.716	.678
25	.912	.860	.808	.759	.711	.666	.620	.574
30	.878	.811	.746	.686	.627	.574	.520	.467
35	.836	.752	.674	.603	.536	.475	.417	.362
40	.783	.682	.592	.512	.439	.375	.316	.262
45	.718	.600	.500	.414	.339	.276	.221	.174

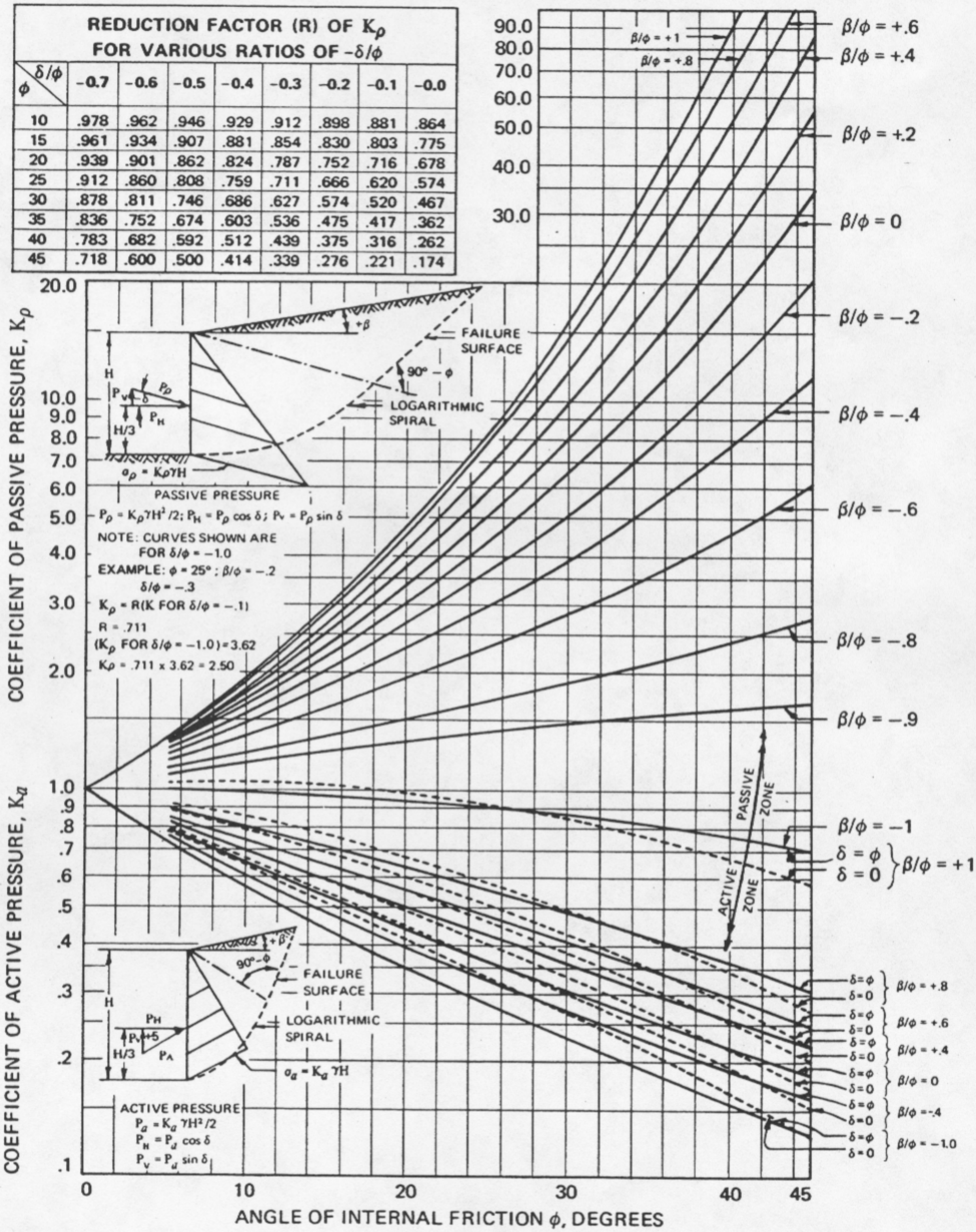


Fig. 5(a) — Active and passive coefficients with wall friction (sloping backfill) (after Caquot and Kerisel[21])

Side 1:

Backfill Slope Angle: $\beta_{s1} := -\mathrm{atan}\left(\frac{1}{2}\right)$ $\beta_{s1} = -26.5651\,\mathrm{deg}$ $\frac{\beta_{s1}}{\phi} = -0.70$

Using the USS Steel Sheet Piling Design Manual, Figure 5(a):

For ϕ = 38 deg and β_s = 0 deg:
$K_a = 0.234$, $K_p = 14.20$, $R_p = 0.773$

For ϕ = 32 deg and β_s = 0 deg:
$K_a = 0.290$, $K_p = 7.85$, $R_p = 0.8366$

For ϕ = 38 deg and β_s = -26.5651 deg:
$K_a = 0.190$, $K_p = 3.060$, $R_p = 0.773$

For ϕ = 32 deg and β_s = -26.5651 deg:
$K_a = 0.230$, $K_p = 1.82$, $R_p = 0.8366$

Active Earth Pressure Coeff: $K_{a1} := 0.190$ USS Fig. 5(a)

Passive Earth Pressure Coeff: $K_{p1} := 3.060$ USS Fig. 5(a)

Reduction for Kp: $R_{p1} := 0.773$ USS Fig. 5(a)

Active Pressure:

$\phi P_{a1} := K_{a1} \cdot \gamma \cdot HC \cdot \phi_{fa}$ $\phi P_{a1} = 21 \frac{\mathrm{psf}}{\mathrm{ft}}$

Passive Pressure:

$\phi P_{p1} := \frac{K_{p1} \cdot R_{p1} \cdot \gamma \cdot HC \cdot Iso \cdot \phi_{fp}}{FS}$ $\phi P_{p1} = 722 \frac{\mathrm{psf}}{\mathrm{ft}}$

Side 2:

Backfill Slope Angle: $\beta_{s2} := -\mathrm{atan}\left(\frac{1}{2}\right)$ $\beta_{s2} = -26.565\,\mathrm{deg}$ $\frac{\beta_{s2}}{\phi} = -0.70$

Active Earth Pressure Coeff: $K_{a2} := 0.190$ USS Fig. 5(a)

Passive Earth Pressure Coeff: $K_{p2} := 3.060$ USS Fig. 5(a)

Reduction for Kp: $R_{p2} := 0.773$ USS Fig. 5(a)

Active Pressure:

$\phi P_{a2} := K_{a2} \cdot \gamma \cdot HC \cdot \phi_{fa}$ $\phi P_{a2} = 21 \frac{\mathrm{psf}}{\mathrm{ft}}$

Passive Pressure:

$\phi P_{p2} := \frac{K_{p2} \cdot R_{p2} \cdot \gamma \cdot HC \cdot Iso \cdot \phi_{fp}}{FS}$ $\phi P_{p2} = 722 \frac{\mathrm{psf}}{\mathrm{ft}}$

Allowable Net Lateral Soil Pressure:

$R_1 := \phi P_{p1} - \phi P_{a2}$ $R_1 = 700 \frac{\mathrm{psf}}{\mathrm{ft}}$ Side 1

$R_2 := \phi P_{p2} - \phi P_{a1}$ $R_2 = 700 \frac{\mathrm{psf}}{\mathrm{ft}}$ Side 2

Depth of Shaft Required:

The function "ShaftD" finds the required shaft depth "d" by increasing the shaft depth until the sum of the moments about the base of the shaft "Msum" is nearly zero. See Figure A for a definition of terms.

$$\mathrm{ShaftD}(d_o, P, R_1, R_2, b, h, y) :=$$

$$\begin{array}{l}
d \leftarrow 0 \cdot ft \\
M_{sum} \leftarrow 100 \cdot lbf \cdot ft \\
\text{while } M_{sum} \geq 0.001 \cdot lbf \cdot ft \\
\quad d \leftarrow d + 0.00001 \cdot ft \\
\quad z \leftarrow \dfrac{2}{d \cdot (R_1 + R_2)} \cdot \left(\dfrac{R_2 \cdot d^2}{2} - \dfrac{R_2 \cdot d_o^{\,2}}{2} - \dfrac{P}{b} \right) \\
\quad x \leftarrow \dfrac{R_2 \cdot z \cdot (d - z)}{R_1 \cdot d + R_2 \cdot (d - z)} \\
\quad P1 \leftarrow (R_2 \cdot d_o) \cdot (d - d_o - z) \\
\quad P2 \leftarrow R_2 \cdot (d - d_o - z)^2 \cdot \dfrac{1}{2} \\
\quad P3 \leftarrow R_2 \cdot (d - z) \cdot x \cdot \dfrac{1}{2} \\
\quad P4 \leftarrow R_1 \cdot d \cdot (z - x) \cdot \dfrac{1}{2} \\
\quad X1 \leftarrow \dfrac{z + d - d_o}{2} \\
\quad X2 \leftarrow \dfrac{2 \cdot z + d - d_o}{3} \\
\quad X3 \leftarrow z - \dfrac{x}{3} \\
\quad X4 \leftarrow \dfrac{1}{3} \cdot (z - x) \\
\quad M_{sum} \leftarrow P \cdot (h + y + d) + b \cdot (-P1 \cdot X1 - P2 \cdot X2 - P3 \cdot X3 + P4 \cdot X4) \\
d
\end{array}$$

Check for 2 load cases. Case 1 has load P acting as shown on Figure A. Case 2 has load P acting in the opposite direction.

Case 1:

$d_{c1} := \mathrm{ShaftD}(d_o, P, R_1, R_2, b, h, y)$ $\qquad d_{c1} = 11.18\,\mathrm{ft}$

$$z_{c1} := \frac{2}{d_{c1}\cdot(R_1+R_2)}\cdot\left(\frac{R_2\cdot d_{c1}^{\;2}}{2}-\frac{R_2\cdot d_o^{\;2}}{2}-\frac{P}{b}\right) \qquad z_{c1} = 5.102\,\mathrm{ft}$$

$$x_{c1} := \frac{R_2\cdot z_{c1}\cdot(d_{c1}-z_{c1})}{R_1\cdot d_{c1}+R_2\cdot(d_{c1}-z_{c1})} \qquad x_{c1} = 1.797\,\mathrm{ft}$$

$$P4_{c1} := R_1\cdot d_{c1}\cdot(z_{c1}-x_{c1})\cdot\frac{1}{2} \qquad P4_{c1} = 12935\,\mathrm{ft}^2\,\frac{\mathrm{psf}}{\mathrm{ft}}$$

Case 2:

$d_{c2} := \mathrm{ShaftD}(d_o, P, R_2, R_1, b, h, y)$ $\qquad d_{c2} = 11.18\,\mathrm{ft}$

$$z_{c2} := \frac{2}{d_{c2}\cdot(R_1+R_2)}\cdot\left(\frac{R_1\cdot d_{c2}^{\;2}}{2}-\frac{R_1\cdot d_o^{\;2}}{2}-\frac{P}{b}\right) \qquad z_{c2} = 5.102\,\mathrm{ft}$$

$$x_{c2} := \frac{R_1\cdot z_{c2}\cdot(d_{c2}-z_{c2})}{R_2\cdot d_{c2}+R_1\cdot(d_{c2}-z_{c2})} \qquad x_{c2} = 1.797\,\mathrm{ft}$$

$$P4_{c2} := R_2\cdot d_{c2}\cdot(z_{c2}-x_{c2})\cdot\frac{1}{2} \qquad P4_{c2} = 12935\,\mathrm{ft}^2\,\frac{\mathrm{psf}}{\mathrm{ft}}$$

Determine Shaft Lateral Pressures and Moment Arms for Controlling Case:

$d := \max(d_{c1}, d_{c2})$ $\qquad d = 11.18\,\mathrm{ft}$

$R_a := \mathrm{if}(d_{c2} \ge d_{c1}, R_1, R_2)$ $\qquad R_a = 700\,\frac{\mathrm{psf}}{\mathrm{ft}}$ $\qquad R_b := \mathrm{if}(d_{c2} \ge d_{c1}, R_2, R_1)$ $\qquad R_b = 700\,\frac{\mathrm{psf}}{\mathrm{ft}}$

$$z := \frac{2}{d\cdot(R_a+R_b)}\cdot\left(\frac{R_a\cdot d^2}{2}-\frac{R_a\cdot d_o^{\;2}}{2}-\frac{P}{b}\right) \qquad z = 5.102\,\mathrm{ft} \qquad x := \frac{R_a\cdot z\cdot(d-z)}{R_b\cdot d+R_a\cdot(d-z)} \qquad x = 1.797\,\mathrm{ft}$$

$$P1 := (R_a\cdot d_o)\cdot(d-d_o-z) \qquad P1 = 1953\,\frac{\mathrm{lbf}}{\mathrm{ft}} \qquad X1 := \frac{z+d-d_o}{2} \qquad X1 = 7.892\,\mathrm{ft}$$

$$P2 := R_a\cdot(d-d_o-z)^2\cdot\frac{1}{2} \qquad P2 = 10901\,\frac{\mathrm{lbf}}{\mathrm{ft}} \qquad X2 := \frac{2\cdot z+d-d_o}{3} \qquad X2 = 6.962\,\mathrm{ft}$$

$$P3 := R_a\cdot(d-z)\cdot x\cdot\frac{1}{2} \qquad P3 = 3825\,\frac{\mathrm{lbf}}{\mathrm{ft}} \qquad X3 := z-\frac{x}{3} \qquad X3 = 4.503\,\mathrm{ft}$$

$$P4 := R_b\cdot d\cdot(z-x)\cdot\frac{1}{2} \qquad P4 = 12935\,\frac{\mathrm{lbf}}{\mathrm{ft}} \qquad X4 := \frac{1}{3}\cdot(z-x) \qquad X4 = 1.102\,\mathrm{ft}$$

$$M_{sum} := P\cdot(h+y+d)+b\cdot(-P1\cdot X1-P2\cdot X2-P3\cdot X3+P4\cdot X4) \qquad M_{sum} = -0.13163\,\mathrm{lbf}\cdot\mathrm{ft}$$

Shaft Design Values:

The Maximum Shear will occur at the bolts or at the top of area 4 on Figure A:

$V_{shaft} := \max\left(P, P4_{c1} \cdot b, P4_{c2} \cdot b\right)$ $\qquad V_{shaft} = 32.34\,\text{kip}$

The Maximum Moment in the shaft will occur where the shear = 0.

Assume that the point where shear = 0 occurs in areas 1 and 2 on Figure A.

<u>Check for Case 1:</u>

$s_{c1} := -d_o + \sqrt{d_o^{\,2} + \dfrac{2 \cdot P}{R_2 \cdot b}}$ $\qquad s_{c1} = 2.808\,\text{ft}$

$M_{shaftc1} := P \cdot \left(h + y + d_o + s_{c1}\right) - R_2 \cdot d_o \cdot b \cdot s_{c1}^{\,2} \cdot \dfrac{1}{2} - R_2 \cdot b \cdot s_{c1}^{\,3} \cdot \dfrac{1}{6}$ $\qquad M_{shaftc1} = 152.1\,\text{kip} \cdot \text{ft}$

Check that the point where shear = 0 occurs in areas 1 and 2 on Figure A:

$Check1 := \text{if}\left[s_{c1} \le \left(d_{c1} - d_o - z_{c1}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad Check1 = \text{"OK"}$

<u>Check for Case 2:</u>

$s_{c2} := -d_o + \sqrt{d_o^{\,2} + \dfrac{2 \cdot P}{R_1 \cdot b}}$ $\qquad s_{c2} = 2.808\,\text{ft}$

$M_{shaftc2} := P \cdot \left(h + y + d_o + s_{c2}\right) - R_1 \cdot d_o \cdot b \cdot s_{c2}^{\,2} \cdot \dfrac{1}{2} - R_1 \cdot b \cdot s_{c2}^{\,3} \cdot \dfrac{1}{6}$ $\qquad M_{shaftc2} = 152.09\,\text{kip} \cdot \text{ft}$

Check that the point where shear = 0 occurs in areas 1 and 2 on Figure A:

$Check2 := \text{if}\left[s_{c2} \le \left(d_{c2} - d_o - z_{c2}\right), \text{"OK"}, \text{"NG"}\right]$ $\qquad Check2 = \text{"OK"}$

$M_{shaft} := \max\left(M_{shaftc1}, M_{shaftc2}\right)$ $\qquad M_{shaft} = 152.09\,\text{kip} \cdot \text{ft}$

Anchor Bolt and Panel Post Design Values:

$V_{bolt} := P$ $\qquad V_{bolt} = 9.36\,\text{kip}$

$M_{bolt} := P \cdot (h + y)$ $\qquad M_{bolt} = 131.04\,\text{kip} \cdot \text{ft}$

Panel Design Value (about a vertical axis):

Find Design Moment for a 1 ft wide strip of wall (between panel posts) for the panel flexure design

$w_{panel} := \max\left[P_w, \max(A \cdot f, 0.1) \cdot \left(4\text{in} \cdot w_c\right)\right]$ $\qquad w_{panel} = 25.0\,\text{psf}$

$M_{panel} := 1.3 \dfrac{w_{panel} \cdot L^2}{8}$ $\qquad M_{panel} = 585 \dfrac{\text{lbf} \cdot \text{ft}}{\text{ft}}$

Panel Post Resistance:

$Cl_{pa} := 1.0in$ Clear Cover to Ties | $h_{pa} := 17in$ Depth of Post

$b_{pa} := 10in$ Width of Post | $bar_A := 10$ Per Design Requirements

Check Flexural Resistance (Std. Spec. 8.16.3):

$\phi_f := 0.90$ Std. Spec. 8.16.1.2.2

$d_{pa} := h_{pa} - Cl_{pa} - dia(3) - \frac{dia(bar_A)}{2}$ $d_{pa} = 14.99\,in$ Effective depth

$A_s := 2 \cdot A_b(bar_A)$ $A_s = 2.54\,in^2$

$a := \frac{A_s \cdot f_y}{0.85 \cdot f'c \cdot b_{pa}}$ $a = 4.48\,in$

$M_n := A_s \cdot f_y \cdot \left(d_{pa} - \frac{a}{2}\right)$ $M_n = 161.91\,kip \cdot ft$

$\phi_f \cdot M_n = 145719\,lbf \cdot ft$

$Check3 := if(\phi_f \cdot M_n \geq M_{bolt}, "OK", "NG")$ $Check3 = "OK"$

Check Maximum Reinforcement (Std. Spec. 8.16.3.1):

$\rho_b := \frac{0.85 \cdot \beta_1 \cdot f'c}{f_y} \cdot \left(\frac{87000 \cdot psi}{87000 \cdot psi + f_y}\right)$ $\rho_b = 0.029$

$\rho := \frac{A_s}{b_{pa} \cdot d_{pa}}$ $\rho = 0.01694$

$Check4 := if(\rho \leq 0.75 \cdot \rho_b, "OK", "NG")$ $Check4 = "OK"$

Check Minimum Reinforcement (Std. Spec. 8.17.1.1):

$S_a := \frac{b_{pa} \cdot h_{pa}^2}{6}$ $S_a = 481.7\,in^3$

$M_{cra} := f_r \cdot S_a$ $M_{cra} = 19.04\,kip \cdot ft$

$Check5 := if(\phi_f \cdot M_n \geq min(1.2 \cdot M_{cra}, 1.33 \cdot M_{bolt}), "OK", "NG")$ $Check5 = "OK"$

Check Shear (Std. Spec. 8.16.6) - Note: Shear Capacity of stirrups neglected:

$\phi_v := 0.85$ Std. Spec. 8.16.1.2.2

$V_{ca} := 2 \cdot \sqrt{\frac{f'c}{psi}} \cdot psi \cdot b_{pa} \cdot d_{pa}$ $V_{ca} = 18.96\,kip$

$Check6 := if(\phi_v \cdot V_{ca} \geq V_{bolt}, "OK", "NG")$ $Check6 = "OK"$

Panel Post Base Resistance:

$b_{pb} := 9in$ Width of Panel Post Base $bar_B := 9$ Per Design Requirements

$h_{pb} := 17.5in$ Depth of Panel Post Base

Check Flexural Resistance (Std. Spec. 8.16.3):

$\phi_f = 0.9$ Std. Spec. 8.16.1.2.2

$d_{pb} := h_{pb} - 0.75in$ $d_{pb} = 16.75\,in$ Effective deptl

$A_s := 2 \cdot A_b(bar_B)$ $A_s = 2\,in^2$

$a := \dfrac{A_s \cdot f_y}{0.85 \cdot f'c \cdot b_{pb}}$ $a = 3.92\,in$

$M_n := A_s \cdot f_y \cdot \left(d_{pb} - \dfrac{a}{2}\right)$ $M_n = 147.89\,kip \cdot ft$

$\phi_f \cdot M_n = 133103\,lbf \cdot ft$

$Check7 := if\left(\phi_f \cdot M_n \geq M_{bolt}, "OK", "NG"\right)$ $Check7 = "OK"$

Check Maximum Reinforcement (Std. Spec. 8.16.3.1):

$\rho_b := \dfrac{0.85 \cdot \beta_1 \cdot f'c}{f_y} \cdot \left(\dfrac{87000 \cdot psi}{87000 \cdot psi + f_y}\right)$ $\rho_b = 0.029$

$\rho := \dfrac{A_s}{b_{pb} \cdot d_{pb}}$ $\rho = 0.01327$

$Check8 := if\left(\rho \leq 0.75 \cdot \rho_b, "OK", "NG"\right)$ $Check8 = "OK"$

Check Minimum Reinforcement (Std. Spec. 8.17.1.1):

$S_b := \dfrac{b_{pb} \cdot h_{pb}^2}{6}$ $S_b = 459.4\,in^3$

$M_{crb} := f_r \cdot S_b$ $M_{crb} = 18.16\,kip \cdot ft$

$Check9 := if\left(\phi_f \cdot M_n \geq \min\left(1.2 \cdot M_{crb}, 1.33 \cdot M_{bolt}\right), "OK", "NG"\right)$ $Check9 = "OK"$

Check Shear (Std. Spec. 8.16.6) - Note: Shear Capacity of stirrups neglected:

$\phi_v = 0.85$ Std. Spec. 8.16.1.2.2

$V_{cb} := 2 \cdot \sqrt{\dfrac{f'c}{psi}} \cdot psi \cdot b_{pb} \cdot d_{pb}$ $V_{cb} = 19.07\,kip$

$Check10 := if\left(\phi_v \cdot V_{cb} \geq V_{bolt}, "OK", "NG"\right)$ $Check10 = "OK"$

Required Splice Length (Std. Spec. 8.25 and 8.32):

Basic Development Length (Std. Spec. 8.25.1):

$$l_{basic}(bar) := \begin{cases} \max\left(\dfrac{0.04 \cdot A_b(bar) \cdot f_y}{\sqrt{\dfrac{f'c}{psi}} \cdot psi \cdot in}, 0.0004 \cdot dia(bar) \cdot \dfrac{f_y}{psi}\right) & \text{if } bar \le 11 \\ \dfrac{0.085 \cdot f_y}{\sqrt{\dfrac{f'c}{psi}} \cdot psi} \cdot in & \text{if } bar = 14 \\ \dfrac{0.11 \cdot f_y}{\sqrt{\dfrac{f'c}{psi}} \cdot psi} \cdot in & \text{if } bar = 18 \\ \text{"error"} & \text{otherwise} \end{cases}$$

$l_{basicA} := l_{basic}(bar_A)$ $\qquad$ $l_{basicA} = 4.02\,ft$

$l_{basicB} := l_{basic}(bar_B)$ $\qquad$ $l_{basicB} = 3.16\,ft$

Development Length (Std. Spec. 8.25):

For top reinforcement placed with more than 12 inches of concrete cast below (Std. Spec. 8.25.2.1):

$l_{dA} := l_{basicA} \cdot 1.4$ $\qquad$ $l_{dA} = 5.62\,ft$

$l_{dB} := l_{basicB} \cdot 1.4$ $\qquad$ $l_{dB} = 4.43\,ft$

Required Lapsplice (Y):

The required lapsplice Y is the maximum of the required lap splice length of bar A (using a Class C splice), the development length of bar B, or 2'-0" per BDM 5.1.2.D.

$LapSplice := \max(1.7 \cdot l_{dA}, l_{dB}, 2 \cdot ft)$ $\qquad$ $LapSplice = 9.56\,ft$

Note: Lap Splices are not allowed for bar sizes greater than 11 per AASHTO Std. Spec. 8.32.1.1.

$Check11 := if(bar_A \le 11 \wedge bar_B \le 11, \text{"OK"}, \text{"NG"})$ $\qquad$ $Check11 = \text{"OK"}$

Anchor Bolt Resistance (Std. Spec. 10.56):

$V_{bolt} = 9360\,lbf$ | $V_{bolt} = 9.36\,kip$

$M_{bolt} = 131040\,lbf \cdot ft$ | $M_{bolt} = 1572.48\,kip \cdot in$

$d_{bolt} := 1.0 \cdot in$ | Per Design Requirements

$$A_{bolt} := \frac{\pi \cdot d_{bolt}^{\,2}}{4}$$ $A_{bolt} = 0.785\,in^2$

$F_t := 30 \cdot ksi$ | Std. Spec. Tbl. 10.56A for A307

$F_v := 18 \cdot ksi$ | Std. Spec. Tbl. 10.56A for A307

$$PanelAxialLoad := \left(4in \cdot \frac{L}{2} + 13in \cdot 10in\right) \cdot (2 \cdot h + y - 3in) \cdot w_c$$ $PanelAxialLoad = 11.959\,kip$

$$f_a := \frac{PanelAxialLoad}{4 \cdot A_{bolt}}$$ $f_a = 3.81\,ksi$ | Axial Compressive Stress

$$f_v := \frac{V_{bolt}}{4 \cdot A_{bolt}}$$ $f_v = 2.98\,ksi$ | Shear Stress

$Check12 := if\left(f_v \le F_v, "OK", "NG"\right)$ | $Check12 = "OK"$

$$f_t := \frac{M_{bolt}}{13.5in} \cdot \frac{1}{2 \cdot A_{bolt}} - f_a$$ $f_t = 70.35\,ksi$ | Tensile Stress

$$F_{t1} := if\left[\frac{f_v}{F_v} \le 0.33, F_t, F_t \cdot \sqrt{1 - \left(\frac{f_v}{F_v}\right)^2}\right]$$ $F_{t1} = 30\,ksi$ | Std. Spec. 10.56.1.3.3

$Check13 := if\left(f_t \le F_{t1}, "OK", "NG"\right)$ | $Check13 = "NG"$

Design Summary:

Wall Height:	$H = 24\,ft$
Required Shaft Depth:	$d = 11.18\,ft$
Maximum Shaft Shear:	$V_{shaft} = 32339\,lbf$
Maximum Shaft Moment:	$M_{shaft} = 152094\,lbf\cdot ft$
Maximum Shaft Moment Accuracy Check (Case 1):	Check1 = "OK"
Maximum Shaft Moment Accuracy Check (Case 2):	Check2 = "OK"
Bar A:	$bar_A = 10$
Post Flexural Resistance (Bar A):	Check3 = "OK"
Maximum Reinforcement Check (Bar A):	Check4 = "OK"
Minimum Reinforcement Check (Bar A):	Check5 = "OK"
Post Shear Check (Bar A):	Check6 = "OK"
Bar B:	$bar_B = 9$
Post Flexural Resistance (Bar B):	Check7 = "OK"
Maximum Reinforcement Check (Bar B):	Check8 = "OK"
Minimum Reinforcement Check (Bar B):	Check9 = "OK"
Post Shear Check (Bar B):	Check10 = "OK"
Lap Splice Length:	LapSplice = 9.558 ft
Lap Splice Allowed Check:	Check11 = "OK"
Bolt Diameter:	$d_{bolt} = 1\,in$
Anchor Bolt Shear Stress Check:	Check12 = "OK"
Anchor Bolt Tensile Stress Check:	Check13 = "NG"

Define Units: $ksi \equiv 1000 \cdot psi$ $kip \equiv 1000 \cdot lbf$ $kcf \equiv kip \cdot ft^{-3}$ $klf \equiv kip \cdot ft^{-1}$

$plf \equiv lbf \cdot ft^{-1}$ $psf \equiv lbf \cdot ft^{-2}$ $pcf \equiv lbf \cdot ft^{-3}$

Design Example 8   Construction Reinforcement in Crossbeam

Material Properties

Height of the crossbeam, h := 48 (in)

Fillet := 3 (in)

Ts := 7.5 (in)

Distance below top construction jo int in Crossbeam, d_joint := 3

Distance from Centroid of Tensile Reinforcment to Farthest Compressive Fiber,

d := h – Fillet − Ts − d_joint (in)

d := 34.5 (in)

Width of Compression Flange, b := 30 (in)

Moment of inertia of the Crossbeam during construction, $I := \left(\frac{1}{12}\right)\cdot b\cdot(h)^3$ I = 276480 (in^4)

Compressive Strength of the Concrete, f'c := 4.00 (ksi)

β1 := 0.85 for 4.00 ksi concrete

Yield Strength of the Reinforcing Stell, fy := 60 (ksi)

Resistance Factor, ϕ := 0.90

Weight of Concrete, wc := 0.160 (kcf)

Bridge Geometry

Length of the Crossbeam, L_crossbeam := 55

Number of Columns supporting the crossbeam, N_col :=2

Col_spacing :=25.0

Critical Moment Location form Left End of Crossbeam, $Crit\_L := \frac{L\_crossbeam}{2} - \frac{Col\_spacing}{2}$

Crit_L = 15ft

Girder Overhang, G_o := 5 (ft)

Girder Spacing, G_s := 7.5 (ft)

Traffic Barrier with unit weight of Wtb := 0.980   klf located at end of crossbeam.

The design moment for construction reinforcement shall be the factored negative dead load moment due to the weight of the crossbeam and adjacent 10 feet of superstructure.

Calculation of Dead Loads of crossbeam and adjacent 10 feet of superstructure.

Distributed weight of crossbeam, $Wc := wc\cdot\left(\frac{h\cdot b}{144}\right)$ Wc = 1.6(klf)

Distributed weight of adjacent 10 feet of slab, $Ws := wc\cdot\left(\frac{Ts}{12}\right)\cdot 12$ Ws = 1 (klf)

A_girder := 2.59 (ft^2)

Weight of Girder, Wg := A_girder·wc·10

Wg = 4.144 (kips)

The Weight of the girder is applied at a distance, $L1 := 10ft$ and $L2 := 2.5ft$ away from the column support. (from bridge Geometry)

Weight of traffic barrier, $Wtb := Wtb \cdot 10$

$Wtb = 9.8$ (kips)

Calculation of Design Moment

Moment due to weight of crossbeam, $Mc := \frac{Crit\_L^2}{2} \cdot Wc$ $\quad Mc = 180$ (kip-ft)

Moment due to weight of slab, $Ms := \frac{Crit\_L^2}{2} \cdot Ws$ $\quad Ms = 112.5$ (kip-ft)

Moment due to Girders, $Mg := Wg \cdot (L1 + L2)$ $\quad Mg = 51.8$ (kip-ft)

Moment due to Traffic Barriers, $Mtb := Wtb \cdot Crit\_L$ $\quad Mtb = 147$ (kip-ft)

Total Factored Negative Design Moment, $Mu := 1.25 \cdot (Mc + Mg + Ms + Mtb)$

$Mu = 614.125$ (kip-ft)

Calculation of Required Steel

Number of Bars, $N\_bars := 6$

Rebar Size, $Bar\_size := 8$

Area of Bars, $As := \left(\frac{Bar\_size}{16}\right)^2 \cdot \pi \cdot N\_bars$ $\quad As = 4.712$ (in^2)

Height of compressive stress block, $a := \frac{fy \cdot As}{0.85 \cdot f'c \cdot b}$ $\quad a = 2.772$ (in)

$Mr := \frac{\phi \cdot As \cdot fy}{12} \cdot \left(d - \frac{a}{2}\right)$ $\quad Mr = 702.207\ kip \cdot ft$ $\quad > Mu = 614.125$ (kip-ft)

Check Cracking Requirements

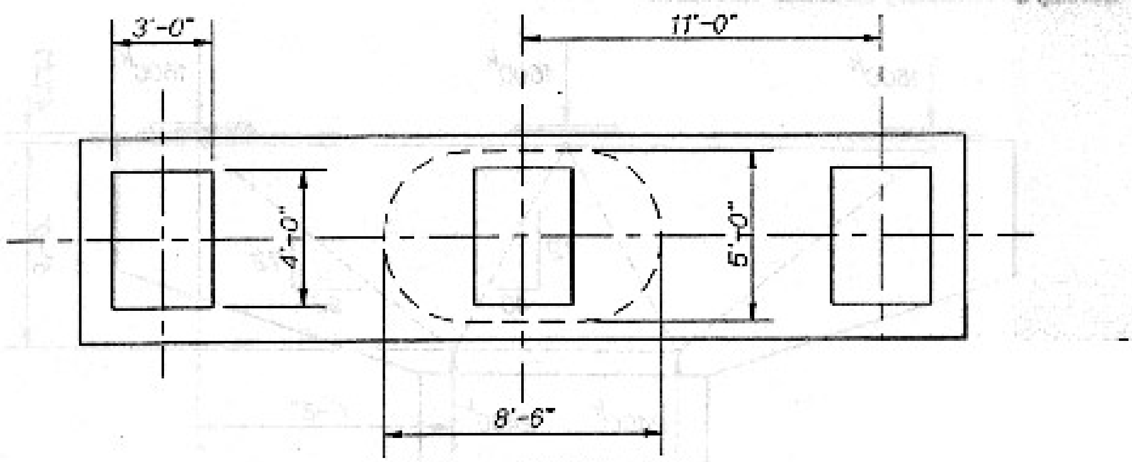
$Yt := \left(\frac{h}{2}\right)$ $\quad Yt = 24$ (in)

$Mcr := \frac{0.24}{12} \cdot \sqrt{f'c} \cdot \frac{I}{Yt}$ $\quad Mcr = 460.8$ (kip-ft)

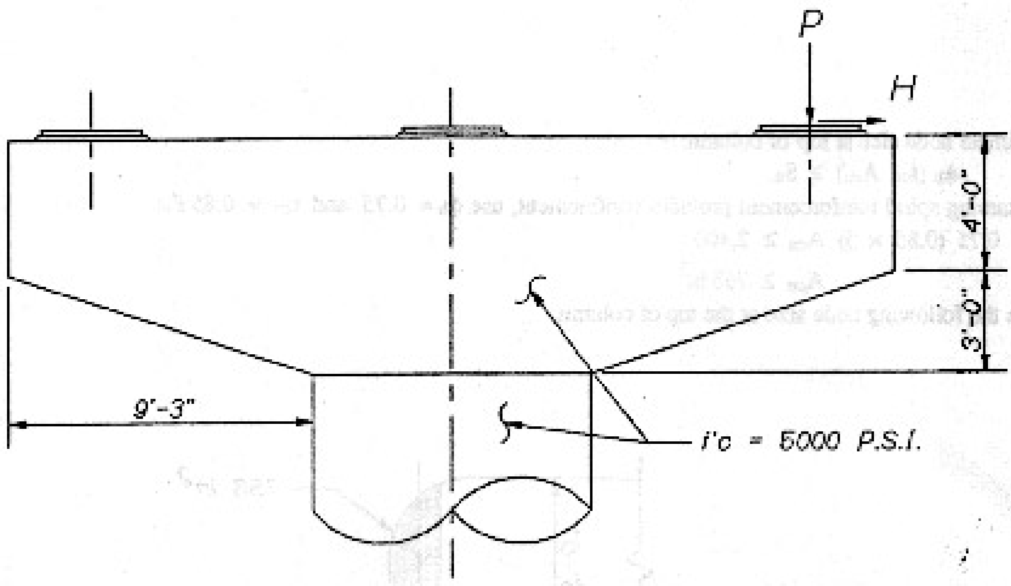
$Factor := 1.2 \cdot Mcr$ $\quad Factor = 552.96$ (kip-ft)

Since Mu >= 1.2 Mcr, the top of the Crossbeam is not going to crack during Construction.

Design Example 9   Stut and Tie Design



PLAN – PIER CAP



ELEVATION – PIER CAP

Design Loads

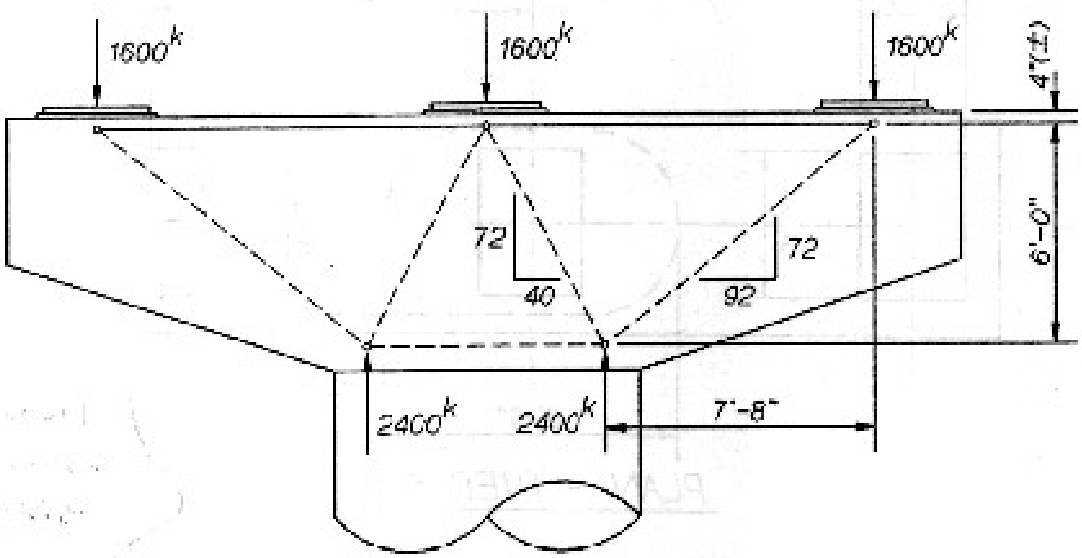
Group I: $P_u = 1600^k$ $H = 0$

Group VII: $P_u = 1500^k$ $H = 400^k$

Assume crossbeam dead load is included with bearing loads.

Use Section 12.4 of *AASHTO's Guide Specifications for Design and Construction of Segmental Concrete Bridges*, 1989.

Develop a Preliminary Strut-and-Tie Model:



STRUT & TIE DIAGRAM

Estimate node size at top of column:

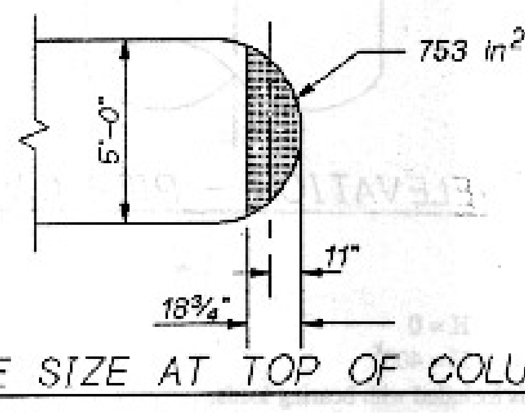
$\phi_b\ (f_{cn}\ A_{cn}) \geq S_u$

Assuming spiral reinforcement provides confinement, use $\phi_b = 0.75$ and $f_{cn} = 0.85\ f'_c$:

$0.75\ (0.85 \times 5)\ A_{cn} \geq 2{,}400$

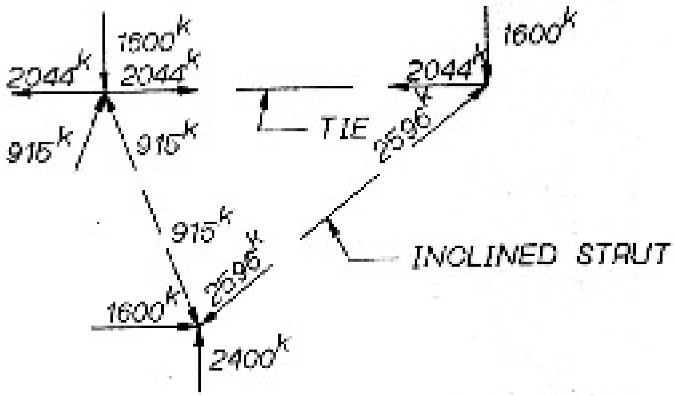
$A_{cn} \geq 753\ in^2$

Use the following node size at the top of column:

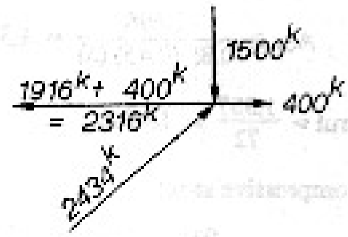


NODE SIZE AT TOP OF COLUMN

Determine Truss Element Forces:



Group I Strut Loads



Group VII Strut Loads

Determine Minimum Sizes of Node Regions:

$\phi_b\ (f_{cn}\ A_{cn}) \geq S_u$ where $\phi_b = 0.70$ for bearing

$f_{cn} = 0.85\ f'_c$ in regions with compression only

$f_{cn} = 0.70\ f'_c$ in regions with one tension tie

At base of inclined strut,

$0.70\ (0.85 \times 5)\ A_{cn} \geq 2{,}596$

$A_{cn} \geq 873\ \text{in}^2$

depth of node $= \dfrac{873}{72''} = 12.1''$ $(72'' \times 12.1'')$

where width of crossbeam = 72″

At top of inclined strut, $A_{cn} \geq \dfrac{2{,}596}{0.70\ (0.70 \times 5)} = \mathbf{1{,}060\ in^2}$

depth of node $= \dfrac{1{,}060}{72''} = 14.7''$ $(72'' \times 14.7'')$

For $1{,}600^k$ chord: $A_{cn} \geq \dfrac{1{,}600}{0.70\ (0.85 \times 5)} = 538\ \text{in}^2$

depth of node $\dfrac{538}{72''} = 7.5''$

For 915^k chord: $A_{cn} \geq \dfrac{915}{1{,}600}\ (538) = 308\ \text{in}^2$

depth of node $\dfrac{308}{72''} = 4.3''$

Determine Minimum Sizes of Compression Members:

$\phi_v\ (f_{cu}\ A_{cs}) \geq S_u$ (inclined compressive struts)

$\phi_f\ (0.85\ f'_c\ A_{cc} + A'_s\ f'_s) \geq S_u$ (compression chords)

For $2{,}596^k$ inclined compressive strut:

$0.85\ (0.45 \times 5)\ A_{cs} \geq 2{,}596^k$ $(f_{cu} = 0.45\ f'_c)$

$$A_{cs} \geq \frac{2{,}596}{0.85\ (0.45)\ (5)} = 1{,}357\ in^2$$

and depth of strut $= \dfrac{1{,}357}{72} = 18.9$ in.

For 915^k inclined compressive strut:

$$A_{cs} \geq \frac{915}{2{,}596}\ (1{,}357) = 478\ in^2$$

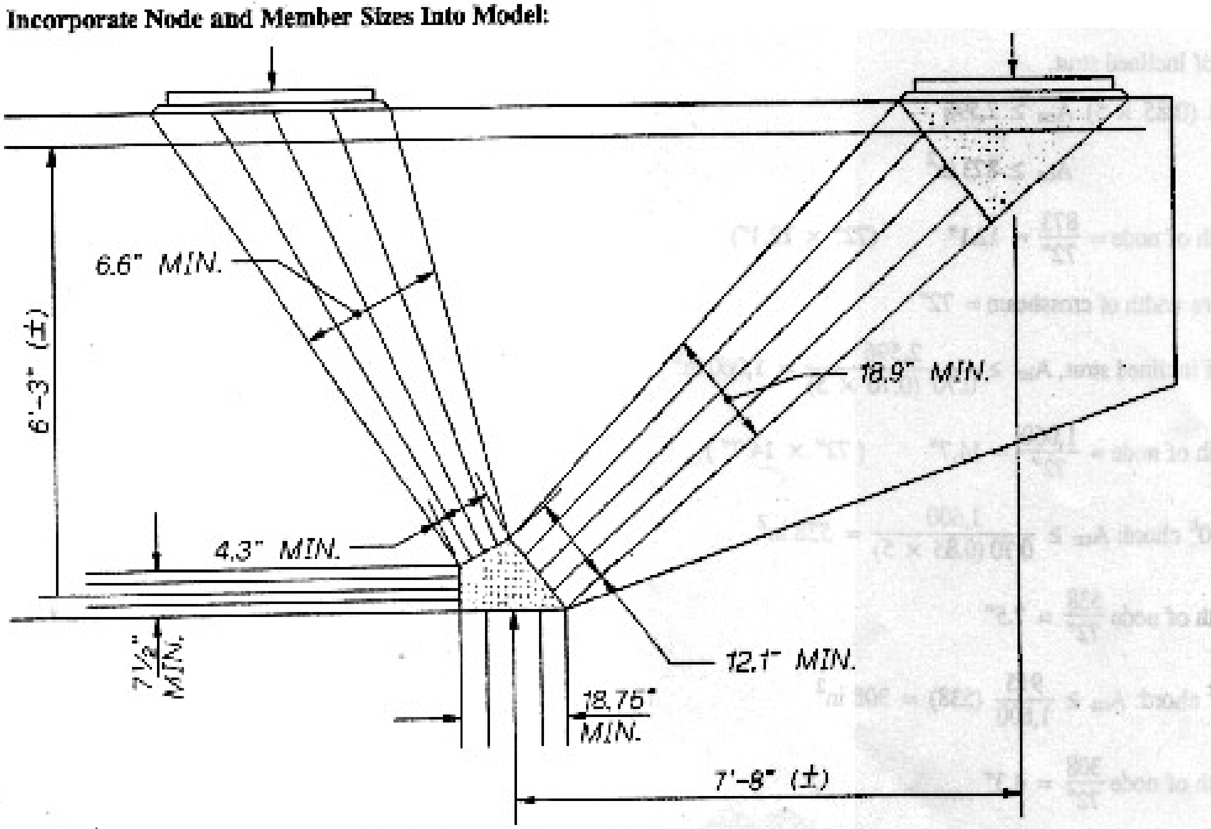
and depth of strut $= \dfrac{478}{72} = 6.6$ in.

For $1{,}600^k$ compression chord:

$$A_{cs} \geq \frac{1{,}600}{0.9\ (0.85)\ (5)} = 418\ in^2$$

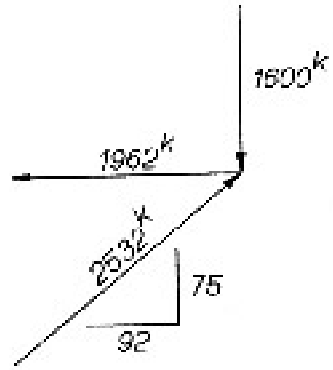
and depth of chord $= \dfrac{418}{72} = 5.8$ in.

Incorporate Node and Member Sizes Into Model:

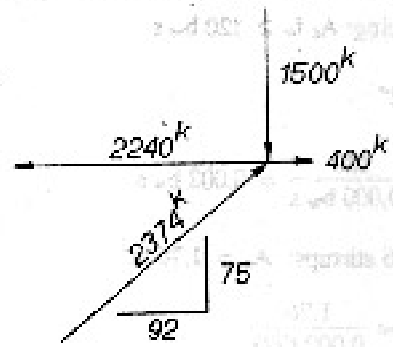


STRUT & NODE GEOMETRY

Recalculate Truss Member Forces:



Group I Strut Loads



Group VII Strut Loads

Design Tie Member:

$\phi_f\ (A_s\ f_{sy}\ +\ A^*_s\ f^*_{su}) \geq S_u$

without prestress: $0.90\ (A_s)\ (60) \geq 2{,}240$

$$A_s \geq 41.5\ in^2$$

Try using 12 bundles of #14 top and #11 bot (A_s = 45.7 in^2)

Check development length of tie bars:

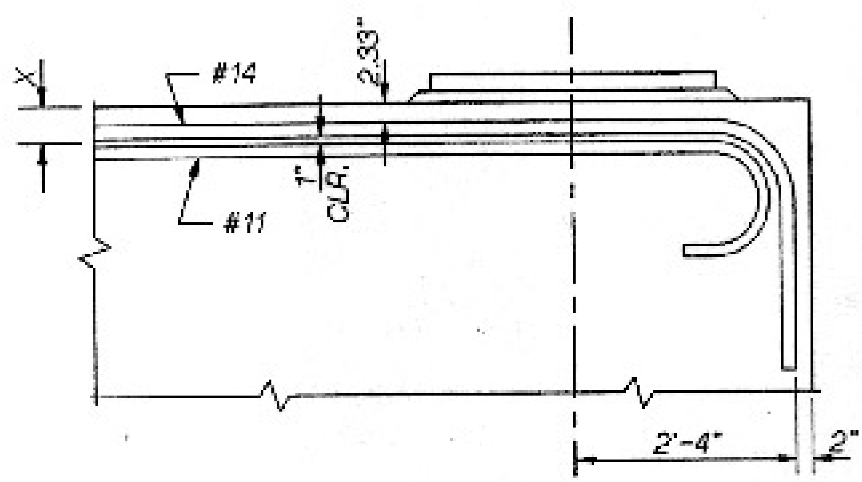
For #14 bars with f'_c = 5,000 psi, l_{dh} = 2′ – 5″

. Development length available = 2′ – 4″ < 2′ – 5″

For #11 bars, l_{dh} = 1′ – 5″ ok

Therefore, total developed steel $A_s = 12\ (1.56)\ +\ 12\ (2.25) \left(\frac{28}{29}\right)$

$$A_s = 44.8\ in^2 > 41.5\ in^2 \quad ok$$



Partial Elevation-Tension Tie at Top of Pier Cap

$$x = \frac{12\ (2.25)\ (3.26)\ +\ 12\ (1.56)\ (5.97)}{45.7} = 4.37'' \approx 4''\ \text{estimate.} \quad ok$$

Determine Minimum Vertical And Horizontal Steel Using Sections 12.5.3.2 and 12.5.3.3.

For vertical reinforcing: $A_s f_y \geq 120 b_w s$

where $s < \dfrac{d}{4}$ or 12″

Therefore, $A_s \geq \dfrac{120}{60{,}000\ b_w\ s} = 0.002\ b_w\ s$

Assume 4 legs of #6 stirrups: $A_s = 1.76\ \text{in}^2$

$$s \leq \frac{A_s}{0.002\ b_w} = \frac{1.76}{0.002\ (72)}$$

$$s \leq 12.2\ \text{in.}$$

Check: $\dfrac{d}{4} = \dfrac{72 - 4.37}{4} = 16.9''$

Therefore, use 4 #6 legs at 12″ maximum spacing.

For horizontal reinforcing: $A_s f_y \geq 120 b_w s$

where $s < \dfrac{d}{3}$ or 12″

For $s = 12''$, $A_s \geq 0.002\ (72)\ (12) = \mathbf{1.73\ in^2}$ (2 – #9 bars)

Try 2 #8 bars: $A_s = 1.58\ \text{in}^2$

$$s \leq \frac{1.58}{0.002\ (72)} = 11.0''$$

Use #8 bars at **11″** maximum spacing on side faces.

For bottom bars, use #6 at approximately 12″ (7 – #6 bars)

Design Example 10 Crossbeam Shear and Torsion Design

Given: Pier Crossbeam with Length, Lc := 35.4ft and with two Columns support, each with centerline 8.5 feet from the end of crossbeam.

Factored Shear, Trosion, and Axial Force:

Vu := 778.3 (kip)

Tu := 722 (kip-ft)

Factored Axial force taken as positive if Tension, Nu := 0 (kips)

Concrete Class 4000. F'c := 4.0 (ksi)

Reinforcing Steel, Grade 60, fy := 60 (ksi)

Modulus of Elasticity, Es := 29000 (ksi)

Unit Weight of Concrete, wc := 0.160 (kcf)

Depth of Crossbeam, D := 48 (in)

Width of Crossbeam, W := 72 (in)

Weight of girder per foot, Wg := D·W·wc $\quad Wg := D \cdot W \cdot \frac{wc}{144} \quad Wg = 3.84$ (kip/ft)

$\frac{Vu}{Lc} = 21.99$ (kips/ft)

$\frac{Tu}{Lc} = 20.4$ (kip-ft/ft)

Calculate the Required Longitudinal Rebar Requirements

Negative Moment Reinforcement

Factored Negative Moment at Columns, Mu := 2500 (kip-ft)

Size of Negative Moment (Top Bar), Barsize_t := 9

Area of one Bar Reinforcement, As :=1 (in^2)

Required Cover for Steel Reinforcing, Cover := 3 (in)

$d := D - Cover - \frac{Barsize\_t}{16} \quad d = 44.44$ (in)

b := W $\quad$ b = 72 (in)

Number of Bars Used For Negative moment reinforcement, Nbars := 14

Total Area of Steel, Ast := Nbars·As

$a := \frac{Ast \cdot fy}{0.85 \cdot f'c \cdot b} \quad a = 3.43$ (in)

Resistance factor to Moment, $\phi := 0.90$

Flexural Resistance, $Mr := \frac{\phi \cdot Ast \cdot fy}{12} \cdot \left(d - \frac{a}{2}\right) \quad Mr = 2691.47$ (kip/ft)

Positive Moment Reinforcement

Factored Negative Moment at Columns, Mu := 1392 (kip-ft)

Size of Negative Moment (Bottom Bar), Barsize_t := 8

Area of one Bar Reinforcement, As := 0.7853 (in^2)

Required Cover for Steel Reinforcing, Cover := 3 (in)

$d := D - Cover - \frac{Barsize\_t}{16}$ $\quad$ d = 44.5 (in)

b := W $\quad$ b = 72 (in)

Number of Bars Used For Negative moment reinforcement, Nbars := 10

Total Area of Steel, Ast := Nbars·As

$a := \frac{Ast \cdot fy}{0.85 \cdot f'c \cdot b}$ $\quad$ a = 1.92 (in)

Resistance factor to Moment, $\phi := 0.90$

Flexural Resistance, $Mr := \frac{\phi \cdot Ast \cdot fy}{12} \cdot \left(d - \frac{a}{2}\right)$ Mr = 1538.55 (kip-ft)

Calculation and Introduction of Torsion Effects

Number of legs of Closed transverse Torsion reinforcing, N_legs := 4

Bar size of Stirrups, Bss := 5

Area of one leg of Closed Transverse Torsion reinforcement, $At := \left(\frac{Bss}{16}\right)^2 \cdot \pi$

Minimum Cover Requirements, Cover := 2.5 $\qquad$ At = 0.307 (in^2)

Perimeter of the centerline of the closed transverse torsion reinforcement, Ph

ph := 2·(w-Cover) +2·(D-cover) $\qquad$ ph = 230 (in)

Area enclosed by centerline of exterior closed transverse torsion reinforcement, including area of holes, if any, Aoh := (W-2·Cover)·(D-2·Cover)

Aoh = 2881 (in^2)

Area enclosed by shear flow path, including any holes, Ao := 0.85·Aoh

Ao = 2448.85 (in^2) $\qquad$ per C5.8.3.6.2 of AASHTO LRFD

For combined shear and torsion, Ɛx shall be determined using Equation 5.8.3.6.2-2 of AASHTO LRFD, with Vu replaced by:

$$Vu := \sqrt{Vu^2 + \left(\frac{0.9 \cdot ph \cdot Tu}{2 \cdot Ao}\right)^2} \qquad Vu = 778.898 \text{ (kips)}$$

Component in the direction of the applied shear of the effective prestressing force; positive if resisting the applied shear, Vp

Since their is no prestressing inthe crossbeam, Vp := 0

Effective Shear Depth, Shall be taken as the greatest of the following per AASHTO LRFD 5.8.2.9

$dv := 0.72 \cdot D$ $\quad dv = 34.56$ (in)

$dv := 0.9 \cdot d$ $\quad dv = 40.05$ (in)

$dv := d - \frac{a}{2}$ $\quad dv = 40.05$ (in)

$dv = 43.54$ (in)

Resistance factor for shear, $\phi := 0.90$

Effective Web Width, $bv := 90$ (inch)

The angle θ shall be as specified in either Table 5.8.3.4.2-1 or Table 5.8.3.6.2-2, as appropriate. with the shear stress, v, taken as for box section:

$$v := \frac{Vu - \phi \cdot Vp}{\phi \cdot bv \cdot dv} + \frac{Tu \cdot ph}{\phi \cdot Aoh^2} \qquad v = 0.243 \text{ (kis)}$$

$\frac{v}{f'c} = 0.061$ $\quad$ Guess a value of $\varepsilon x := .001$

From Table 5.8.3.4.2-1 of AASHTO LRFD,

$\theta := 36.4$ (degrees)

$\beta := 2.23$

Calculate εx from Equation 5.8.3.4.2-1 of AASHTO LRFD and compare to guess.

Since there is no prestressing inthe Crossbeam:

$Aps := 0$ (in^2)

$fpo := 0$ (ksi)

$Eps := 285000$ (ksi)

$$\varepsilon x := \frac{\frac{Mu}{dv} + 0.5 \cdot Nu + \frac{0.5 \cdot (Vu - Vp)}{\tan\left(\frac{\pi}{180} \cdot \theta\right)} - Aps \cdot fpo}{2 \cdot (Es \cdot As + Eps \cdot Aps)} \qquad \varepsilon x = 0.0123$$

Try again with a new value of $\varepsilon x = .00125$

$\theta := 38.6$ (degrees)

$\beta := 2.09$

$$\varepsilon x := \frac{\frac{Mu}{dv} + 0.5 \cdot Nu + \frac{0.5 \cdot (Vu - Vp)}{\tan\left(\frac{\pi}{180} \cdot \theta\right)} - Aps \cdot fpo}{2 \cdot (Es \cdot As + Eps \cdot Aps)} \qquad \varepsilon x = 0.01141$$

The nominal Shear resistance, Vn, shall be determined as the lesser of:

$Vn = Vc + Vp + Vs$

$Vn = 0.25\ f'c\ bv\ dv + Vp$

$Vn\_1 := \frac{Vu}{\phi}$ $Vn\_1 = 865.44$ (kips)

$Vn\_2 := 0.25 \cdot f'c \cdot bv \cdot dv + Vp$ $Vn\_2 = 3918.39$ (kips)

$Vn := Vn\_1$ $Vn = 865.442$ (kips)

Shear Resistance Provided by Concrete, $Vc := 0.0316\sqrt{f'c \cdot bv \cdot dv}$ $Vc = 247.64$ (kips)

Shear Resistance Provided by Stirrups, $Vs := Vn - Vc - Vp$ $Vs = 617.8$ (kips)

$Av := At \cdot N\_legs$ $Av = 1.227\ (in^2)$

Required Spacing of Transverse Reinforcing, $s := \frac{Av \cdot fy \cdot dv}{\tan\left(\theta \cdot \frac{\pi}{180} \cdot Vs\right)}$ $s = 6.5$ (in)

Check Spacing Requirements

Check the minimum Transverse Reinforcement per AASHTO LRFD 5.8.2.5.

$Av := 0.0316 \cdot \sqrt{f'c} \cdot \frac{bv \cdot s}{fy}$ $Av = 0.616$ (in^2)

Steel Provided, $As\_prov := At \cdot N\_legs$ $As\_prov = 1.227\ (in^2)$ OK

Check the maximum spacing of the transverse reinforcement per AASTHO LRFD 5.8.2.7

If $v_u < 0.125\ f'c$ then:

$s_{max} = 0.8d_v \le 24$ (in) (5.8.2.7-1)

If $v_u \ge 0.125\ f'c$ then:

$s_{max} = 0.4d_v \le 12$ (in) (5.8.2.7-2)

The supplied spacing meets the minimum required spacing.

The nominal torsional resistance shall be taken as per AASHTO LRFD 5.8.3.6.2:

$Tn := \frac{\phi \cdot 4 \cdot Ao \cdot At \cdot fy}{s \cdot \tan\left(\theta \cdot \frac{\pi}{180}\right)}$ $Tn = 31274.33$(kip/in)

The transverse reinforcing for shear meets the requirements for the required torsional resistance.

Check Longitudinal Reinforcement

$$Asfy := \frac{Mu}{\phi \cdot dv} + \frac{0.5 \cdot Nu}{\phi} + \frac{1}{\tan\left(\theta \cdot \frac{\pi}{180}\right)} \cdot \left[\sqrt{\left(\frac{Vu}{\phi} - 0.5 \cdot Vs - Vp\right)^2 + \left(\frac{0.45 \cdot ph \cdot Tu}{2 \cdot Ao \cdot \phi}\right)^2}\right]$$

$(Ast + Asb) \cdot fy = 1311.18$ (kips) $\ge$ $Asfy = 733.016$ (kips) OK

Design Example 11 Torsion and Shear Capacity of Reinforced Concrete Beams

Concrete Properties:

$f'c := 4 \cdot ksi$

Reinforcement Properties:

Bar Diameters: Bar Areas:

$$dia(bar) := \begin{vmatrix} 0.375 \cdot in \text{ if } bar = 3 \\ 0.500 \cdot in \text{ if } bar = 4 \\ 0.625 \cdot in \text{ if } bar = 5 \\ 0.750 \cdot in \text{ if } bar = 6 \\ 0.875 \cdot in \text{ if } bar = 7 \\ 1.000 \cdot in \text{ if } bar = 8 \\ 1.128 \cdot in \text{ if } bar = 9 \\ 1.270 \cdot in \text{ if } bar = 10 \\ 1.410 \cdot in \text{ if } bar = 11 \\ 1.693 \cdot in \text{ if } bar = 14 \\ 2.257 \cdot in \text{ if } bar = 18 \end{vmatrix} \qquad A_b(bar) := \begin{vmatrix} 0.11 \cdot in^2 \text{ if } bar = 3 \\ 0.20 \cdot in^2 \text{ if } bar = 4 \\ 0.31 \cdot in^2 \text{ if } bar = 5 \\ 0.44 \cdot in^2 \text{ if } bar = 6 \\ 0.60 \cdot in^2 \text{ if } bar = 7 \\ 0.79 \cdot in^2 \text{ if } bar = 8 \\ 1.00 \cdot in^2 \text{ if } bar = 9 \\ 1.27 \cdot in^2 \text{ if } bar = 10 \\ 1.56 \cdot in^2 \text{ if } bar = 11 \\ 2.25 \cdot in^2 \text{ if } bar = 14 \\ 4.00 \cdot in^2 \text{ if } bar = 18 \end{vmatrix}$$

$f_y := 40 \cdot ksi$

$E_s := 29000ksi$ LRFD 5.4.3.2

$E_p := 28500ksi$ LRFD 5.4.4.2 for strands

$bar_{LT} := 18$ Longitudinal -Top

$bar_{LB} := 18$ Longitudinal -Bottom

$bar_T := 6$ Transverse

$d_{LT} := dia(bar_{LT})$ $d_{LT} = 2.257in$ $A_{LT} := A_b\,(bar_{LT})$ $A_{LT} = 4in^2$

$d_{LB} := dia(bar_{LB})$ $d_{LB} = 2.257in$ $A_{LB} := A_b\,(bar_{LB})$ $A_{LB} = 4in^2$

$d_T := dia(bar_T)$ $d_T = 0.75in$ $A_T := A_b\,(bar_T)$ $A_T = 0.44in^2$

Beam Section Properties:

$b := 37in$

$h := 90in$

$bottomcover := 1.625 \cdot in$

$sidecover := 1.625 \cdot in$

$topcover := 1.625 \cdot in$

Factored Loads:

$V_u := 450 \cdot kip$

$T_u := 500 \cdot kip \cdot ft$

$M_u := 0 \cdot kip \cdot ft$

$N_u := 0 \cdot kip$

Torsional Resistance:

The factored Torsional Resistance shall be: LRFD 5.8.2.1

$Tr = \phi \cdot T_n$

Torsion shall be investigated where:

$T_u > 0.25 \cdot \phi \cdot T_{cr}$

$\phi := 0.90$ For Torsion and Shear LRFD 5.5.4.2

$A_{cp} := b \cdot h$ $A_{cp} = 3330 in^2$

$P_c := (b + h) \cdot 2$ $P_c = 254 in$

$f_{pc} := 0 \cdot ksi$

$$T_{cr} := 0.125 \cdot \sqrt{\frac{f'c}{ksi}} \cdot \frac{\left(\frac{A_{cp}}{in^2}\right)^2}{\left(\frac{P_c}{in}\right)} \cdot \sqrt{1 + \frac{\frac{f_{pc}}{ksi}}{0.125 \cdot \sqrt{\frac{f'c}{ksi}}}} \cdot kip \cdot in$$

$T_{cr} = 10914 kip \cdot in$

$T_{cr} = 909.5 kip \cdot ft$

$0.25 \cdot \phi \cdot T_{cr} = 204.6 kip \cdot ft$

$T_u > 0.25 \cdot \phi \cdot T_{cr} = 1$ Torsion shall be investigated.

For a section subjected to combined Shear and Torsion:

$$p_h := 2 \cdot \left[\left[b - 2 \cdot \left(sidecover + \frac{d_T}{2} \right) \right] + \left(h - topcover - bottomcover - d_T \right) \right]$$

$p_h = 238 in$

$$A_{oh} := \left[b - 2 \cdot \left(sidecover + \frac{d_T}{2} \right) \right] \cdot \left(h - topcover - bottomcover - d_T \right)$$

$A_{oh} = 2838 in^2$

$A_o := 0.85 \cdot A_{oh}$ $A_o = 2412.3 in^2$

Adjusted Shear for calculation of ε:

$$V_{ust} := \sqrt{V_u^2 + \left(\frac{0.9 \cdot p_h \cdot T_u}{2 \cdot A_o} \right)^2}$$

$V_{ust} = 522.9 kip$ LRFD 5.8.3.6.2-2

$\theta := 36.4 \cdot deg$ Assume to begin iterations

$d_e := 90in - 30in$ $d_e = 87in$ LRFD 5.8.2.9

$d_v := \max(0.9 \cdot deg, 0.72 \cdot h)$ $d_v = 78.3in$

$b_v := b$ $b_v = 37in$

$V_p := 0 \cdot kip$ No Prestress Stands

$A_{ps} := 0 \cdot in^2$ No Prestress Stands

$A_s := 4 \cdot A_{LB}$ $A_s = 16in^2$ For 4 #18 bars

$f_{po} := 0 \cdot ksi$

$$\varepsilon_x := \frac{\left[\frac{M_u}{d_v} + 0.5 \cdot N_u + 0.5 \cdot (V_{ust} - V_p) \cdot \cot(\theta) - A_{ps} \cdot f_{po}\right]}{E_s \cdot A_s + E_p \cdot A_{ps}}$$ LRFD 5.8.3.4.2-2

$\varepsilon_x \cdot 1000 = 0.764$

$$v_u := \sqrt{\left(\frac{V_u - \phi \cdot V_p}{\phi \cdot b_v \cdot d_v}\right)^2 + \left(\frac{T_u \cdot p_h}{\phi \cdot A_{oh}^{\ 2}}\right)^2}$$ LRFD 5.8.3.6.2-4

$v_u = 0.262\,ksi$

$\frac{v_u}{f'c} = 0.065$

From Table 5.8.3.4.2-1, Find β and θ

$\theta := 36.4 \cdot deg$ Value is close to original guess. OK

$\beta := 2.23$

Torsional Resistance:

$A_t := A_T$ $A_t = 0.44in^2$

$s := 5 \cdot in$ Spacing of Reinforcement resisting Torsion

$$T_n := \frac{2 \cdot A_o \cdot A_t \cdot f_y \cdot \cot(\theta)}{s}$$ $T_n = 23035\ kip \cdot in$ LRFD 5.8.3.6.2-1

$T_n = 1920\ kip \cdot ft$

$T_r := \phi \cdot T_n$ $T_r = 20731\ kip \cdot in$ LRFD 5.8.2.1

$T_r = 1728\ kip \cdot in$

$T_r \geq T_u = 1$ OK

Shear resistance:

The factores Shear Resistance shall be: LRFD 5.8.2.1

$V_r = \phi \cdot V_n$

$$V_c := 0.0316 \cdot \beta \cdot \sqrt{\frac{f'c}{ksi}} \cdot b_v \cdot d_v \cdot ksi$$ $V_c = 408.3\ kip$ LRFD 5.8.3.3

$\alpha := 90 \cdot deg$

$A_v := A_T$ $\quad$ $A_v = 0.44 in^2$

$s := 5 \cdot in$ $\quad$ Spacing of Reinf. resisting Shear

$$V_s := \frac{A_v \cdot f_y \cdot d_v \cdot (\cot(\theta) + \cot(\alpha)) \cdot \sin(\alpha)}{s} \qquad V_s = 373.8 \text{ kip}$$

$V_n := \min(V_c+V_s+V_p,\ 0.25 \cdot f'c \cdot b_v \cdot d_v+V_p)$ $\quad$ $V_n = 782.1$ kip

$V_r := \phi \cdot V_n$ $\quad$ $V_r = 703.9$ kip

$V_r \geq V_u = 1$ $\quad$ OK

Check for Longitudinal Reinforcement: LRFD 5.8.3.6.3

$f_{ps} := 0 \cdot ksi$

$A_s \cdot f_y + A_{ps} \cdot f_{ps} = 640.0$ kip

$$\frac{M_u}{\phi \cdot d_v} + \frac{0.5 \cdot N_u}{\phi} + \cot(\theta) \cdot \sqrt{\left(\frac{V_u}{\phi} - 0.5 \cdot V_s - V_p\right)^2 + \left(\frac{0.45 \cdot p_h \cdot T_u}{2 \cdot A_o \cdot \phi}\right)^2} = 469.7 \text{ kip}$$

640.0 kip >= 469.7 kip $\quad$ OK

Design Example 12 Strain Compatibility Approach for Flexural Capacity

Materials: Cast- in-place Slab

Actual thickness, tc = 8.0in

Structural thickness, height_slab :=7.5in

Concrete Strength at 28 days, f'c_slab := 4.0ksi

Precast Beams: AASHTO - PCI Bulb Tee

Concrete Strength at 28 days, f'c_beam := 6.5ksi

t_top_flange := 42in

Prestressing Strands: 1/2" Diameter, Seven Wire, Low Relaxation

Area of One Strand, Area_bars 0.153in^2

fpu := 270ksi

Yield Strength, fpy := 0.9fpu fpy = 243ksi =

Stress Limits for prestressing strands:

At Service Limit State (after all losses) , fse < 0.80fpy = 194.4ksi

Modulus of Elasticity, Eps := 28500ksi

Total Number of Prestressing Strands, num_bars := 48

Centroid of Prestressing Strands above bottom of Girder, center_prestressing_strands 6.9in

Non Composite Section Properties

h_beam := 72in

Step 1 Assume a neutral axis depth c and substitute in the equation below to obtain the corresponding strain in each steel layer "i". A layer "i" is defined as a group of bars or tendons with the same stress-strain properties and effective prestress, and which can be assumed to have a combined area with a s ingle centroid.

di = Depth of steel from Extreme compression fiber (in)

fse = effective prestress. For partially tensioned tendons or for non-reinforced rebars, f_{se} may be assumed = f_{pi} - 25 ksi where f_{pi} is initial tension.

c := 9.75in fse := 194.4ksi

center_prestressing_strands = 6.9in di := (h_beam+height_slab)-center_prestressing_strands

$$\varepsilon si := 0.003 \cdot \left(\frac{di}{c} - 1 \right) + \frac{fse}{Eps} \qquad \varepsilon si = 0.026$$

Step 2 Estimate the stress in each steel layer

For Low Relaxation 270 ksi Strands: $fsi := fpu - \frac{0.04ksi}{(\varepsilon si - 0.007)}$ $fsi = 267.91\,ksi$

Step 3 Use Equilibrium of Forces to Check assumed Neutral Axis Depth

$\Sigma Asifsi + \Sigma Fcj = 0$

$\Sigma Asifsi := (Area\_bars \cdot num\_bars) \cdot fsi \qquad \Sigma Asifsi = 1967.5\,kip$

Average the coefficient β1 over the concrete materials if the depth of the compression block is greater than the depth of the cast in place concrete slab.

$$\beta_{1avg} = \frac{\Sigma(f'c\ Ac\ \beta 1)j}{\Sigma(f'c\ Ac)_j}$$

initial guess of $\beta 1 := 0.81$ $\quad$ $initial\_a := \beta 1\ c$ $\quad$ $initial\_a := 7.897in$ $\quad$ $width\_of\_slab := 60in$

$$\beta 1avg := \frac{f'c\_slab \cdot width\_of\_slab \cdot height\_slab \cdot 0.85 + f'c\_beam \cdot (initial\_a - height\_slab) \cdot t\_top\_flange \cdot 0.725}{f'c\_slab \cdot width\_of\_slab \cdot height\_slab + f'c\_beam \cdot (initial\_a - height\_slab) \cdot t\_top\_flange}$$

$\beta 1avg = 0.843 \qquad a := \beta 1avg \cdot c \qquad a = 8.22in$

$$\Sigma Fcj := -0.85 \cdot (width\_of\_slab \cdot height\_slab) \cdot f'c\_slab - 0.85 \cdot (a - height\_slab) \cdot t\_top\_flange \cdot f'c\_beam$$

$$\Sigma Fcj = -1696.66kip$$

$\Sigma Fcj + \Sigma Asifsi = 270.89kip$ $\qquad$ Error is not with in 1% so repeat Procedure

Step 4 Revise "c" and repeat steps 1 through 3 as necessary By decreasing the value of "c", the area of the compressive stress block decreases. This corresponds to a much smaller compressive force. If the value of "c" is increased, the compressive force increases. After a few trials, a neutral axis that satisfied equilibrium was found. For simplicity, all intermediate iterations will be skipped.

Step 1 Final Trial

Assume new neutral axis at $c := 11.30in$ and compute strain in each stell layer.

$$\varepsilon si := 0.003 \cdot \left(\frac{di}{c} - 1\right) + \frac{fse}{Eps} \qquad \varepsilon si = 0.023$$

Step 2 Final Trial

$$fsi := fpu - \frac{0.04ksi}{(\varepsilon si - 0.007)} \qquad fsi = 267.51$$

Step 3 Final Trial

$\Sigma Asifsi := Area\_bars \cdot num\_bars \cdot fsi \qquad \Sigma Asifsi = 1964.63kip$

$\beta 1 := 0.824 \qquad initial\_a := \beta 1 \cdot c \qquad initial\_a = 9.31in$

Note: The new Value of β1 is equal to the value of β1avg of the last iteration.

$$\beta 1avg := \frac{f'c\_slab \cdot width\_of\_slab \cdot height\_slab \cdot 0.85 + f'c\_beam \cdot (initial\_a - height\_slab) \cdot t\_top\_flange \cdot 0.725}{f'c\_slab \cdot width\_of\_slab \cdot height\_slab + f'c\_beam \cdot (initial\_a - height\_slab) \cdot t\_top\_flange}$$

$\beta 1avg = 0.823 \qquad a := \beta 1avg \cdot c \qquad a = 9.3in$

$$\Sigma Fcj := -0.85 \cdot (width\_of\_slab \cdot height\_slab) \cdot f'c\_slab - 0.85 \cdot (a - height\_slab) \cdot t\_top\_flange \cdot f'c\_beam$$

$$\Sigma Fcj = -1947.83kip$$

$\Sigma Fcj + \Sigma Asifsi = 16.8kip$ $\qquad$ Error is with in 1%, so equilibrium is satisfied.

Step 5 Calculate the nominal flexural capacity by summing moments of all forces about the top fiber of the compressive face.

di = distance from top of slab to the midpoint of pretensioned steel   di = 72.6in

dj1 = distance from top of slab to centroid of compressive block on slab,

$$dj1 := \frac{height\_slab}{2} \qquad dj1 = 3.75\,in$$

dj2 = distance from top of slab to centroid of compressive block on beam,

$$dj2 := height\_slab + \frac{a - height\_slab}{2} \qquad dj2 = 8.4\,in$$

$Fcj1 := 0.85 \cdot width\_of\_slab \cdot height\_slab \cdot f'c\_slab$ $Fcj1 = 1530kip$

$Fcj2 := 0.85 \cdot (a - Height\_slab) \cdot t\_top\_flange \cdot f'c\_beam$ $Fcj2 = 417.83kip$

$\Sigma Fcjdj := Fcj1 \cdot dj1 + Fcj2 \cdot dj2$ $Mn := \Sigma Asifsi \cdot di - \Sigma Fcjdj$ $Mn = 11115kip \cdot ft$

Step 6 Calculate the Design Moment Capacity, ϕMn

$\phi := 1$ For Precast Concrete Fleural Members

$Mr := \phi \cdot Mn$

$Mr := 11115kip \cdot ft$

	AASHTO LRFD Specifications	Strain Compatibility
Neutral Axis Depth, c (in)	12.14	11.30
Compression Block Depth, a (in)	10.32	9.30
$\phi M\nu$	10782kip-ft	Mr =11115 kip·ft
	97%	100%